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Journal of the

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CONTENTS

December, 1959

Papers

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THE VORTEX CHAMBER AS AN AUTOMATIC FLOW-CONTROL DEVICE

R. C. Kolf,[1] M. ASCE and P. B. Zielinski,[2] A. M. ASCE

SYNOPSIS

Coefficients of discharge for flow through horizontal orifices are greatly educed when a vortex is present. Several flow control devices have been deigned utilizing this phenomenon. This paper presents a rational explanation f the resulting coefficient variation from dynamic similarity. The results of aboratory experiments are shown, giving a means of predicting orifice flow haracteristics from the inlet velocity condition.

INTRODUCTION

The free vortex has long been recognized as a significant phenomenon in e design of hydraulic structures. For the most part, however, it has been egarded as an undesirable possibility, and its consideration has been limited the design of expensive appurtenances intended to eliminate, or at least reice, the circular movements of the fluid.

In recent years, the properties of the free vortex have been utilized sucessfully to solve several flow control problems. Heim(3) made studies of a counterflow brake" for use in pumping installations. This device consists of spiral shaped chamber with tangential and axial connections. The normal rection of flow is in at the center (axial) connection and out at the peripheral angential) connection. If the flow is reversed due to power failure, the relting tangential inlet conditions cause a vortex motion within the chamber d therefore a greater resistance to flow. This device is thought of as a type pipe fitting which may replace the check valve in special cases.

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Stevens[5] made practical use of the vortex phenomenon in the design o diversion structure for a combined sewer system. A small dam in the se at the downstream side of the diversion structure diverted all of the low s tary flow into an orifice chamber. The radial approach to the horizontal c fice resulted in normal orifice head-discharge relations. The high flow r in the main sewer for a storm condition would pass over the dam, causing entrance to the diversion chamber to be tangential and a vigorous vortex t form. The resultant reduced orifice coefficient of discharge allowed only small portions of the storm water to pass into the interceptor.

Both of these control devices exhibit the advantage of relatively simple struction and maintenance because they have no moving parts. It should b expected that these ideas would find more popular use in hydraulic design cept for the fact that to date very little information of design value is avai ble concerning the characteristics of orifice flow with a significant vortex present.

Theory

The following signs and symbols are used in this paper and have been l here for easy reference.

A,	area of orifice
B,	diameter of boundary ring
C,	orifice coefficient of discharge
D,	diameter of orifice
f_c,	centrifugal force per unit volume
f_i,	inertia force per unit volume
g,	acceleration due to gravity
H,	static head on center of orifice
p,	pressure
R,	Reynolds number
u,	tangential component of velocity vector
v,	discharge component of velocity vector
V,	velocity vector
$\underline{\underline{V}}$,	vortex number
x, y,	co-ordinates of a point in the flow field
α,	angle the divergent jet makes with the horizontal
Γ,	circulation
ρ,	mass density
μ,	dynamic viscosity

It has been shown by dimensional analysis[5] that if the variables affec flow through a horizontal orifice are known to be

$$f\,(D, B, p, \rho, V, \mu, \Gamma) = 0$$

hen the discharge equation will take the form

$$Q = CA\sqrt{2gH} \qquad (1)$$

$$C = f\left(\frac{B}{D}, R, \frac{\Gamma}{VD}\right) \qquad (2)$$

Γ, is the circulation for the free vortex flow and is defined

$$\Gamma = 2\pi x u \qquad (3)$$

The drawdown of the water surface resulting from this vortex motion is (see Fig. 1)

$$y = \frac{u^2}{2g} \qquad (4)$$

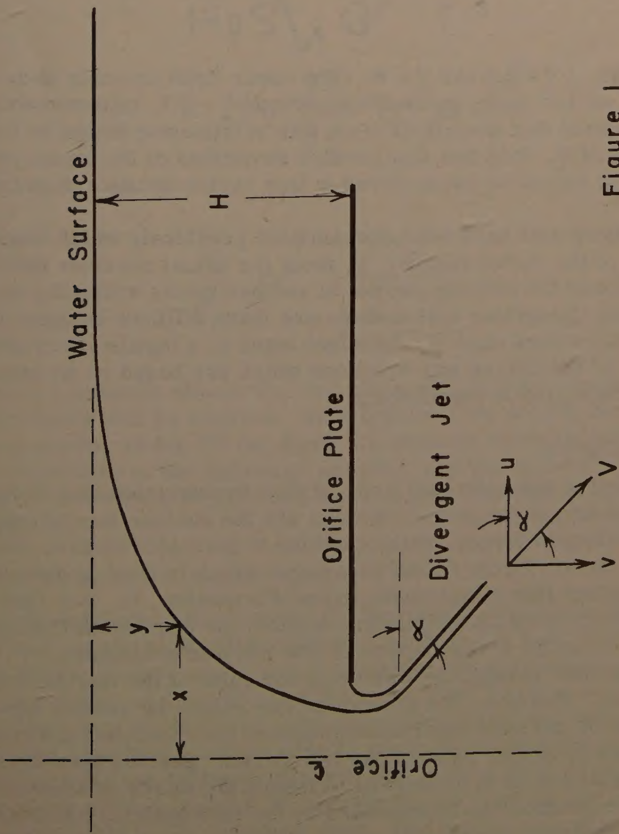

Figure 1

$$y = \frac{\Gamma^2}{8\pi^2 x^2 g}$$

The resulting water surface profile is hyperbolic. Superimposing radi flow on this vortex motion does not change the shape of the curve except i regions immediately near the orifice, because both components of the res tant velocity vary inversely with the radius.

In the earlier studies made at the University of Wisconsin,[5] it was no that for vortex conditions strong enough to alter the orifice coefficient of charge, the angle which the velocity vector makes with the radius, is such the radial component is of negligible value in computing the drawdown.

By studying the water surface profile, and computing the circulation, Γ from Eq. (5), the coefficient of discharge in Eq. (2), was found to be

$$C = 0.686 - 0.218\underline{\underline{V}}$$

$$\underline{\underline{V}} = \frac{\Gamma}{D\sqrt{2gH}}$$

between limits of $\underline{\underline{V}} = 0.8$ and $\underline{\underline{V}} = \pi$. The upper limit amounts to an extra lation, since no data were accumulated beyond $\underline{\underline{V}} = 2.7$, approximately. It might be expected that deviations from this relationship would be found at higher values of $\underline{\underline{V}}$. It is felt that greater deviations of the actual profile the theoretical should be encountered in this region because of greater sh effects.

No laboratory data have been accumulated previously which would allo computation of the vortex number, $\underline{\underline{V}}$, from the actual chamber flow patter Measurements of the velocity profile at various points within the orifice chamber using immersion type meters are made difficult because of their turbance of the vortex motion. This fact leads to a legitimate criticism o design value of the curves and equations which are based on an assumed velocity-surface profile relationship.

Dynamic Similarity

In studying the dynamic similarity of flow through metering devices wh the forces governing the particle motion are the surface forces represent by p and the viscous forces, which combine to give accelerative affects in cated by the inertia force, f_i, the parameter which is used to determine si larity of different flow cases is the Reynold's number, R. For flow throu particular device such as the pipeline orifice, the coefficient C does not v greatly with R except in the regions of low Reynold's numbers.

This parameter is described as being the ratio of the inertial forces to viscous forces. However, the division of one vector by another has no kno mathematical or physical significance; hence, the relationship so express is the absolute or scalar ratio of the two forces. Besides insuring simila of force magnitudes, it is necessary to insure similarity of kinematic effe (velocity, acceleration) as represented by the flow pattern, which amounts accounting for the vector aspects of the problem. Ordinarily this is accor plished by specifying the geometric similarity with particular values of th

atio of orifice diameter to pipeline diameter. This is permissible since for given boundary configuration, the flow pattern is uniquely defined (if a sufficient approach length, measured usually in terms of orifice diameter, is pecified in order to insure uniform flow in the approach to the orifice. This s another geometric effect.)

In the case of orifice flow with the possibility of tangential components to he velocity vectors, similarity cannot be defined in terms of the scalar teynold's number and the boundary geometry as indicated by $\frac{B}{D}$, because for a articular total acceleration as indicated by the Bernoulli equation solution,

$$V = \sqrt{2gH} \tag{8}$$

here is an infinite number of possibilities for the direction of the vectors. This problem can be handled by breaking the total inertial force into components and specifying

$$f_c = \rho \frac{u^2}{x} \propto \rho \frac{\Gamma^2}{L^3} \tag{9}$$

$$f_i = \rho V \frac{dV}{ds} \propto \rho \frac{V^2}{L} \tag{10}$$

ie ratio

$$\frac{f_c}{f_i} \propto \frac{\Gamma^2}{L^2 V^2} \tag{11}$$

ill determine kinematic similarity, and along with B/D (geometric), and R, ynamic similarity will be assured. This ratio will be seen to be the square f the vortex number in Eq. (2) or, since the velocity which is characteristic f the inertial effects is the discharge velocity, and the value of Γ/L which is haracteristic of the centrifugal component is Γ/D (Eq. (3)), it is also the quare of Eq. (7).

xperimental Methods and Results (Fig. 3)

The experimental work was done in the Marquette University Hydraulics aboratory and consisted of tests made in a sheet metal tank 5 feet square and feet high. Orifices were cut in 2-foot diameter, 14 gage brass plates and iounted in the center of the bottom of the tank. Two inner boundary rings ere used. Each was constructed of 14 gage sheet metal and had two diameically opposite openings 6 inches wide to which were attached parallel sheet ietal plates forming the entrance channel, which were fixed in position makg an arbitrarily chosen angle of 30 degrees (or 20 degrees for some of the sts) with the tangent to the boundary ring. The larger ring was 3.5 feet in

diameter and 2.5 feet high and the smaller ring was 2.0 feet in diameter 2.5 feet high. Water was admitted to the tank through 2 pipes 2 inches in ameter each of which branched near the floor of the tank into perforated inch pipes which distributed the inflow uniformly around the outer bounda Vortex profile measurements were taken as a check on the earlier Unive of Wisconsin tests(5) with a moving point gage mounted on an aluminum b 5 feet long with a 2-1/2 inch by 5/8 inch cross section. Discharge was measured by a 3-inch orifice in a 4-inch pipeline upstream from the test For the low-flow tests a 2-inch nutating disc type water meter was used.

In the current series, 64 test runs were made in which the head was va from 0.3 to 1.8 feet, $\underline{\underline{V}}$ varied from 0.85 to 2.6, and C varied from 0.2 to 0

For the initial tests the larger boundary ring was used and the entranc channel vanes set at a 30 degree angle with the boundary tangent. Tracer and a small current meter were used to check for a uniform velocity with the entrance channels. No significant variation in velocity from the insid outside of the channels was found; therefore, the average velocity was co puted from the discharge and cross-sectional area, and this value was us further computations. Within the boundary ring the velocity was found to essentially tangential (negligible radial components except immediately n the orifice) and its magnitude the same as the entrance velocity. The cha in direction of the fluid velocity vector had thus been accomplished with n appreciable change in momentum indicating negligible shear forces. For some of the tests, both entrance channels were used, and for others one w sealed shut. The results in either case were identical even though some centricity of the air core was noted for the one vane condition.

The results for the 20 degree approach angle exactly corresponded to 30 degree angle. The velocity within the boundary (at a 1.75 foot radius f the center of the orifice) was found to be the same as that within the entra vanes, and so this value was used in computing the circulation from Eq. (

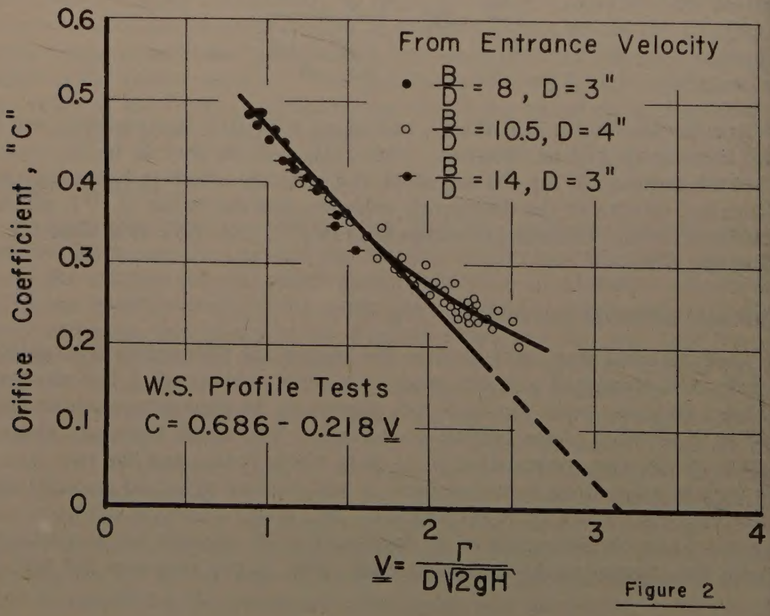

Figure 2

The curve (Fig. 2) shows the variation of orifice coefficient with vortex number for these data as computed from the entrance velocity. The data closely follow the straight line indicated by the water surface profile measurements for vortex numbers less than 2.0. Above this value a separate curve is evidently established. This deviation probably occurs because the profile computations were based on a measurement made only one diameter away from the orifice, more closely approximating the true energy condition in that region. High vortex numbers represent conditions of high velocity near the orifice and therefore greater shear losses as already noted.

As a further check on the normal acceleration relations, measurements of the angle α (Fig. 1) of the divergent jet were made. From Eq. (3)

$$\frac{\Gamma}{D} = \pi u \tag{12}$$

(because upon release, the circular motion becomes tangential) Substitution in Eq. (7) gives

$$\underline{\underline{V}} = \pi \frac{u}{V} = \pi \cos \alpha \tag{13}$$

$$\cos \alpha = \frac{\underline{\underline{V}}}{\pi} \tag{14}$$

Results of these measurements are shown below.

$\underline{\underline{V}}$	$\cos^{-1} (\underline{\underline{V}}/\pi)$	α (measured)
0.952	72.4°	70°
1.25	66.6°	64°
1.70	57.3°	60°
1.86	53.7°	52.5°
2.24	44.5°	46°
21.25	44.3°	49°

Figure 3

Experimental Apparatus

Because of the small clearance between the tank bottom and the floor of Hydraulic Laboratory, considerable difficulty was experienced in making t measurements. A complete series of such tests has thus not yet been atte ed. Because the jet was not well defined (considerable wavering) the checl indicated by these data is better than had been expected.

CONCLUSIONS

Prediction of orifice coefficients for vortex flow can be made on the ba of the velocity at inlet to the orifice chamber. The accuracy of such predi for chambers which are concentric with the orifice should depend on the al ty to estimate the inlet velocity. The curves shown should be extended to clude larger vortex numbers. To do this it may be necessary to build an a paratus in which the head and inlet velocity can be independently controlle (such as a model of the Stevens combined sewer system which affords the vantage for laboratory purposes of by-passing part of the total flow.

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THE SETTLING PROPERTIES OF SUSPENSIONS

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ABSTRACT

This paper contains the results of an analytical and experimental investigation of the settling properties of suspensions of particles in fluid. The use of these properties in predicting the sedimentation of the particles is outlined.

1. INTRODUCTION

In many engineering problems it is necessary to deal with a flowing fluid in which particles are suspended. The engineer commonly encounters such problems in river channels, in water and sewage clarification, in reservoirs and in the delta regions of rivers. One of the tasks associated with these problems is to determine how much material will settle out of suspension and where that material will settle.

The factors that affect the settling of suspended particles can be divided into two groups. Those of the first group can be called the conditions of flow. They are the temperature and pressure in the suspension, the velocity distribution of the flow, and the nature of the turbulence of flow. The factors of the second group can be called the settling properties of the suspension.

The term "settling properties of a suspension" refers to how the particles behave in a given set of conditions of flow. Thus, for some specified temperature, pressure, fluid velocity and turbulence, the particles of a suspension will settle, flocculate or be diffused in some manner. The manner will vary from suspension to suspension, and must be determined experimentally for each.

Hence, the problem of determining how material will settle out of a flowing suspension can be divided into two parts. First the flow conditions must be

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measured, and second, the settling properties of the suspension must be sured. Neither of these two is an easy task, and considerable research development is necessary before either will be done satisfactorily.

This paper deals only with the settling properties of a suspension and use in calculating the removal of the particles. Section 2 contains a gen discussion of the problem of calculating the removal. This discussion po out how the settling properties are to be used and indicates the type of ex mental information needed. The experimental determination of settling perties is discussed in Section 3. Some results already obtained are pre ed, and the experiments yet to be developed are outlined. In Section 4, th research is related to more practical engineering problems.

2. Calculating the Removal of Suspended Particles

a. The Problem

It was stated above that one of the tasks of the engineer is to determi advance how the particles will settle out of a given flowing suspension. specific, consider a suspension of particles in water which is flowing in open channel. The fluid velocity and turbulence level are low enough for particles to settle to the bottom of the channel. The problem is to calcul the amount of settled material as a function of distance along the channel

Such a calculation involves the solution of some form of continuity eq expressing the conservation of suspended matter at any point in the susp In order to derive the equation it is first necessary to consider the facto fecting the concentration of particles at a point. The first of these facto the settling of the particles.

b. Settling of the Suspended Particles

The term "settling velocity of a suspended particle" refers to the vel that the particle would have if it were settling in perfectly still fluid. A pension may contain particles of many settling velocities and the distrib of these settling velocities is an important settling property of the suspe In calculations related to settling it is just as reasonable to characterize ticles by these settling velocities as by their size, shape, and density. F is the settling velocity that indicates how soon the particle is going to be moved from suspension by settling to the bottom.

Let w represent settling velocity and let it be positive downward. Th locities of the particles in a suspension will range from zero to the velo of the fastest particle. For numerical calculations and theoretical discu it is convenient to divide the range of values of w into a number of incre Δw. Then w_1 can refer to a class of settling velocities between the valu and Δw, and w_2 to those velocities between Δw and $2\Delta w$. In general, w_i indicate velocities between the values $(i - 1)\,\Delta w$ and $i\Delta w$. The particles velocity w_i will be called i-particles.

The concentration of i-particles at any point in the suspension will be called f_i, the units of which are mass per unit volume of suspension. Th concentration can vary from point to point and from time to time. Lettin y, and z represent coordinates which locate a point, and t represent time concentration will be a function, $f_i\,(x, y, z, t)$. The term f_i indicates that function which varies as i varies. Hence, f is the frequency function for

ling velocities at point x,y,z. In this paper it will be called the settling ve-
ty distribution at x,y,z.
At any point in the suspension, the flux of i-particles due to settling will be
en by

$$w_i\, f_i\,(x, y, z, t)$$

s is the rate at which i-particles pass through a unit horizontal area at
nt x,y,z, and time t. The rate at which all particles pass through this area

$$\sum_{i=1}^{\infty} w_i\, f_i\,(x, y, z, t)$$

By definition, this total flux is simply the product of the local instantaneous
an settling velocity, $\overline{w}(x,y,z,t)$, and the local instantaneous particle concen-
tion, $\phi(x,y,z,t)$. That is,

$$\overline{w}(x, y, z, t)\, \phi\,(x, y, z, t) = \sum_{i=1}^{\infty} w_i\, f_i\,(x, y, z, t) \qquad (1)$$

ere

$$\phi\,(x, y, z, t) = \sum_{i=1}^{\infty} f_i \qquad (2)$$

ce, the product $\overline{w}\phi$ describes the motion of particles due to settling.

Movement of Particle Due to Fluid Motion

At any point in the suspension, the particles also have a motion due to fluid
v at the point. If the flow is without turbulence, this motion is simply the
ultant fluid velocity. Let this resultant be represented by the vector
,y,z,t). It will have x,y, and z-components of U(x,y,z,t), V(x,y,z,t) and
,y,z,t) respectively.
When the flow of a suspension is turbulent, the motion of the particles is
ally considered in two parts. First, the particles are considered as being
ried along by the temporal mean fluid velocity, also called $\underline{U}$(x,y,z,t).
erimposed on this motion is the diffusion of particles by the turbulent
tuations of fluid velocity. The decision as to what is the main flow and
t is turbulence will depend upon the flow pattern for each individual case.
In order to deal with the particle motion caused by turbulence, it is common
define a coefficient of diffusion. Let e_{ik} be the coefficient for i-particles
x direction. Then the flux of i-particles through a unit area normal to the
rection can be given as

$$-e_{ik}\,\frac{\partial f_i}{\partial x}$$

ce there may also be an e_{iy} and e_{yz}, the coefficient will be called e_i when
specific direction is implied.
The coefficient e_i should not be confused with the coefficient for diffusion
transfer of momentum between neighboring elements of fluid. For the sake

of differentiation the latter will be called e_m. A reasonable assumption, made, is that e_i is equal to e_m. For a detailed discussion of this point, t reader is referred to the experimental and analytical work of Vanoni[1] a Ismail.[2] Their work indicates that e_i not only differs e_m, but that the manner in which it differs varies with i. For coarse sand (w_i large) in f water, e_i was smaller than e_m while for fine sand (w_i small) e_i was larg Nevertheless, the order of magnitude of the difference was such that it m be reasonable to assume that $e_i = e_m$ in engineering calculations. With t assumption there is no need for the subscript i and the diffusion coeffici for particles can be called e with an appropriate subscript when specific rections are mentioned.

Vanoni also found that e appears to vary with the particle concentratic As yet, there is so little information on this variation that it cannot be cc sidered in the calculation of removal. However, it must be an important fect when ϕ is large. It is hoped that future research will yield quantitat information about the effect of ϕ on e. In this paper, the effect is ignore

d. Equation of Continuity

The motion of particles due to settling, mean temporal fluid velocity a turbulent diffusion produces a flux of particles in one or all of the x,y, a directions. The resultant flux can be represented by the vector. In orde describe the vector let $\underline{i}$, $\underline{j}$ and $\underline{k}$ be unit vectors in the x,y and z-directi respectively and let z be positive in the direction of positive w.

Consider first the flux of i-particle. The resultant is represented by vector, $\underline{a}_i$, where

$$\underline{a}_i (x, y, z, t) = \underline{k}\, w_i f_i + \underline{U} f_i - \underline{i}\, e_{ix} \frac{\partial f_i}{\partial x} - \underline{j}\, e_{iy} \frac{\partial f_i}{\partial y} - \underline{k}\, e_{iz} \frac{\partial f_i}{\partial z}$$

The conservation of i-particles at x,y,z,t dictates that the divergence of $\underline{a}$ plus the time rate of change of f_i minus any source flow of i-particles zero. Hence,

$$\nabla \cdot \underline{a} + \frac{\partial f_i}{\partial t} - P_i (x, y, z, t) = 0$$

where

$$\nabla = \underline{i} \frac{\partial}{\partial x} + j \frac{\partial}{\partial y} + \underline{k} \frac{\partial}{\partial z}$$

The term, P_i, in Eq. (3) represents a distributed source of i-particle y,z. This term accounts for the effects of hindered settling and floccula If a particle of class w_i becomes attached to another in the process of fl lation it will generally experience a change in settling velocity. Hence, appears from class w_i and appears in another class. Similarly a particl have one settling velocity when the local concentration has a certain valu when the local concentration changes, hindered settling may change the v locity to w_i. It follows that P_i must represent the rate at which particle quire settling velocity w_i less the rate at which i-particles acquire other locities. The units of P_i are mass per unit time per unit volume.

For an incompressible fluid the expansion of Eq. (3) becomes

$$\frac{\partial f_i}{\partial t} + \frac{\partial(w_i f_i)}{\partial z} + U\frac{\partial f_i}{\partial z} + V\frac{\partial f_i}{\partial y} + W\frac{\partial f_i}{\partial z} - \frac{\partial}{\partial x}\left(e_{ix}\frac{\partial f_i}{\partial x}\right)$$

$$- \frac{\partial}{\partial y}\left(e_{iy}\frac{\partial f_i}{\partial y}\right) - \frac{\partial}{\partial z}\left(e_{iz}\frac{\partial f_i}{\partial z}\right) - P_i = 0 \qquad (5)$$

ccept for the last term this equation is similar to that expressed by bbins,(3) McNown,(4) Van Driest,(5) and others.

There is an Eq. (5) for each value of i. By summing these equations and roducing Eqs. (1) and (2) into the sum, the continuity equation for total ncentration becomes

$$\frac{\partial \phi}{\partial t} + \frac{\partial(\bar{w}\,\phi)}{\partial z} + U\frac{\partial \phi}{\partial x} + V\frac{\partial \phi}{\partial y} + W\frac{\partial \phi}{\partial z} - \frac{\partial}{\partial x}\left(\sum_i e_{ix}\frac{\partial f_i}{\partial x}\right)$$

$$- \frac{\partial}{\partial y}\left(\sum_i e_{iy}\frac{\partial f_i}{\partial y}\right) - \frac{\partial}{\partial z}\left(\sum_i e_{iz}\frac{\partial f_i}{\partial z}\right) - \sum_i P_i = 0 \qquad (6)$$

Since flocculation and hindered settling produce no mass,

$$\sum_i P_i = 0 \qquad (7)$$

bstituting Eq. (7) into (6) and assuming that e_i is the same for all i gives the lowing equation:

$$\frac{\partial \phi}{\partial t} + U\frac{\partial \phi}{\partial x} + V\frac{\partial \phi}{\partial y} + W\frac{\partial \phi}{\partial z} + \frac{\partial}{\partial z}(\bar{w}\,\phi) - \frac{\partial}{\partial x}\left(e_x\frac{\partial \phi}{\partial x}\right)$$

$$- \frac{\partial}{\partial y}\left(e_y\frac{\partial \phi}{\partial y}\right) - \frac{\partial}{\partial z}\left(e_z\frac{\partial \phi}{\partial z}\right) = 0 \qquad (8)$$

The effects of flocculation and hindered settling do not appear explicitly in . (8). These effects do appear implicitly, however, because they cause anges in $\bar{w}$. Eq. (8) cannot be solved without information on how $\bar{w}$ will ange with particle concentration, flocculation, differential settling, and difsion. This information will be obtained from studies of the settling propers of the suspension. In short, the settling properties of the suspension ocr implicitly in Eq. (8) in the term $\bar{w}$.

Initial and Boundary Conditions

The solution of Eq. (8) requires a knowledge of initial conditions or bounda- conditions or both. It becomes necessary, therefore, to think about specific ysical situations. For present purposes, consider the section of open chan- l shown in Fig. 1. The distance along the channel is called x and is taken positive in the direction of flow. The distance down from the free surface the suspension is called z and is positive downward. The y-direction is rmal to x and z in such a way as to produce a right-handed xyz system.

For this channel, the initial condition is the settling velocity distribution in plane x = 0. Hence, it is necessary to take a sample of suspension from a nt at x = 0 and obtain the settling velocity distribution of the sample by exrimental analysis. If the distribution is not constant over the channel crossction, samples must be taken from several points in the section and each

sample must be analyzed. The result will be a spatial distribution of set velocity distribution at x = 0, and this result is the initial condition for se in the channel.

The boundary conditions occur at the top surface of the suspension and the bottom of the channel. At the top, the condition is the rate at which p cles cross the surface. Usually no particles pass through this surface. in a density current along the bottom of a reservoir or settling tank, mat may be settling out of relatively still water above into the current below.

At the bottom of the channel, the boundary condition depends on the be of the particles which have already settled to the bottom. If particles wh once settled to the bottom of the channel are picked up into the flow, the process is called entrainment or resuspension. Let the rate at which pa cles are picked up per unit area of bed be E. The boundary condition can be stated as

$$E = -\left(e_z \frac{\partial \phi}{\partial z}\right)_{\text{at bed}}$$

This equation is of the form used by Dobbins[3] to describe pickup of uni particles in a turbulence jar. Dobbins verified this equation experimenta for E = constant and E = 0.

Theoretical work by Lane and Kalinske[6] shows that E will depend up the settling velocities of the settled particles, the roughness of the bed a pattern of turbulence near the bed. Thus for a given flow and bed conditi E will depend on the properties of the suspension.

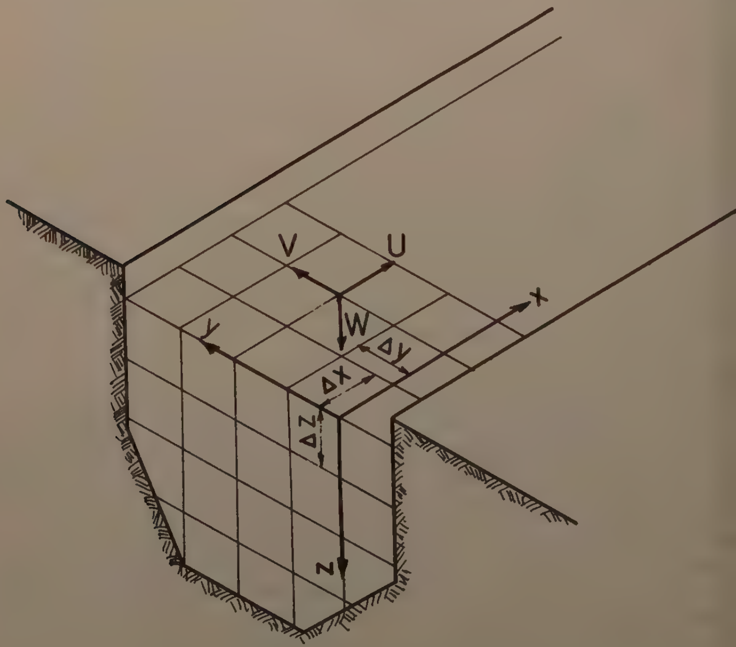

Fig. I. Suspension flowing in open channel.

The resuspension of particles must be studied experimentally for each sus-
sion. During the study, the flow conditions should be varied and resus-
sion measured. The end result should give the resuspension rates as a
tion of flow conditions for the suspension being studied. Experiments of
sort are currently in progress at the California Institute of Technology.
ppears that a great deal of research is still to be done before the results
be used in calculating removal.

olution of the Continuity Equation

Except for a few simple cases it is necessary to solve the equation of con-
ity by some numerical method. For this purpose, Eq. (8) is written in the
n

$$\frac{\partial \phi}{\partial t} + \left(U - \frac{\partial e_x}{\partial x}\right)\frac{\partial \phi}{\partial x} + \left(V - \frac{\partial e_y}{\partial y} + \left(W - \frac{\partial e_z}{\partial z}\right)\frac{\partial \phi}{\partial z}\right.$$

$$+ \frac{\partial(\bar{w}\phi)}{\partial z} - e_x\frac{\partial^2 \phi}{\partial y^2} - e_y\frac{\partial^2 \phi}{\partial y^2} - e_z\frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (10)$$

n Eq. (10), U, V, W, e_x, e_y and e_z are all determined by measurements. In
eral, they will be functions of x, y, z. However, if turbulence is affected by
icle concentration, the e's will also be a function of ϕ. In such a case the
tionship between e and ϕ will have to be determined experimentally.
Eq. (10) is in a form easily reduced to finite-difference form. For this
pose, the space occupied by the suspension is divided into rectangular
llelpipeds as shown at the upstream end of the channel in Fig. 1. Each
llelepiped has the dimensions Δx, Δy and Δz as shown. The corners of
parallelepipeds form a lattice of points where the value of ϕ is to be found.
The subscript ℓ will indicate that a point in the lattice has an x-coordinate
Δx, while subscripts m and n indicate coordinate $m\Delta y$ and $n\Delta z$ in the y-
z-directions respectively. Using this notation, Eq. (10) can be written in
following form, for steady state settling.

$$\left(U - \frac{\partial e_x}{\partial x}\right)_{\ell,m,n} \frac{\phi_{\ell+1,m,n} - \phi_{\ell,m,n}}{\Delta x}$$

$$+ \left(V - \frac{\partial e_y}{dy}\right)_{\ell,m,n} \frac{\phi_{\ell,m+1,n} - \phi_{\ell,m,n}}{\Delta y}$$

$$+ \left(W - \frac{\partial e_z}{\partial z}\right)_{\ell,m,n} \frac{\phi_{\ell,m,n+1} - \phi_{\ell,m,n}}{\Delta z}$$

$$+ \frac{(\bar{w}\phi)_{\ell,m,n+1} - (\bar{w}\phi)_{\ell,m,n}}{\Delta z}$$

$$- (e_y)_{\ell,m,n} \frac{\phi_{\ell+1,m,n} - 2\phi_{\ell,m,n} + \phi_{\ell-1,m,n}}{\Delta x^2}$$

$$- (e_y)_{\ell,m,n} \frac{\phi_{\ell,m+1,n} - 2\phi_{\ell,m,n} + \phi_{\ell,m-1,n}}{\Delta y^2}$$

$$- (e_z)_{\ell,m,n} \frac{\phi_{\ell,m,n+1} - 2\phi_{\ell,m,n} + \phi_{\ell,m,n-1}}{\Delta z^2} = 0$$

To use Eq. (11) it is necessary to know ϕ and $\overline{w}$ at all points in the $x = r = 0$. This knowledge constitutes the initial condition. Any resusp of particles between the planes $x = 0$ and $x = \Delta x$ constitutes the bounda dition and must also be known. With these known quantities, it is possi calculate ϕ at all points in the plane $x = \Delta x$ or $r = 1$.

In order to repeat the process, it is necessary to know $\overline{w}\phi$ at points plane $r = 1$. Hence, the fundamental part of this calculation is to deter how $\overline{w}\phi$ changes between the plane $r = 0$ and $r = 1$, or, in general, betw two planes, r and $r + 1$. This change in $\overline{w}\phi$ will depend on the settling ties of the suspension.

Once the ϕ is calculated over cross sections at various values of r simple matter to compute the rate at which particles are removed in a distance, $r\Delta x$. Thus the problem stated in part (a) above can be solvec

The procedure outlined here requires more information than is usu available to the engineer. Its main value, at present, is in showing spe ly what research and development are necessary for complete calculat Methods of measuring fluid velocity and turbulent diffusion must be im methods of measuring resuspension must be developed, and methods s the settling properties of a suspension must be expanded.

Furthermore, the finite-difference calculation shows that informati the settling properties of a suspension must be obtained in a very spec form. This information must relate changes in the local mean settling $\overline{w}(x,y,z)$ to particle settling, particle concentration, flocculation, hinde turbulence and any other conditions prevailing in the immediate neighb of the point x,y,z. A program of research was initiated for the purpos taining this kind of information about suspensions. The progress mad is reported in Section 3.

Due to the problems involved in calculating removal, it is natural t of studying settling in hydraulic scale models. However, the model st be misleading unless the similarity of settling is considered. Such si is discussed next.

g. Scale Models and Similarity in Sedimentation

The relationship between settling in the model and the prototype is tained by reducing Eq. (8) to dimensionless form. All velocities in th can be given in terms of a characteristic velocity U_0 and dimensionle velocities U*, V* and W*. The result is

$$U = U^* U_o, \quad V = V^* V_o, \quad W = W^* W_o, \quad w_i = w_i^* U_o$$

ilarly, distance can be written in terms of a characteristic length x_o, and usion coefficients can be written in terms of a characteristic coefficient, as follows:

$$y = y^* y_o, \quad y = y^* y_o, \quad z = z^* z_o$$

$$e_x = e_x^* e_o, \quad e_y = e_y^* e_o, \quad e_z = e_z e_o$$

imensionless concentration can also be written in terms of a character- c value, ϕ_o.

$$\phi^* = \frac{\phi}{\phi_o}$$

nensionless time can be written in terms of characteristic length and ve- ity.

Substituting these dimensionless quantities in Eq. (8) gives

$$\frac{\partial \phi^*}{\partial t^*} + U^* \frac{\partial \phi^*}{\partial x^*} + V^* \frac{\partial \phi^*}{dy^*} + W^* \frac{\partial \phi^*}{dz^*} + \frac{\partial (\overline{w}^* \phi^*)}{dz^*}$$

$$+ \frac{e_o}{U_o x_o} \quad \frac{\partial}{\partial y^*} (e_x^* \frac{\partial \phi^*}{\partial x^*}) + \frac{\partial}{\partial y^*} (e_y^* \frac{\partial \phi^*}{\partial y^*}) + \frac{\partial}{\partial z^*} (e_z^* \frac{\partial \phi^*}{\partial z^*}) = 0 \tag{12}$$

There are three kinds of similarity involved in Eq. (12). The first two are metric and kinemetric. Because of them the quantity $\frac{e_o}{U_o x_o}$ will be the ne dimensionless constant in model and prototype. Furthermore, the value $\overline{w}/U_o$ at a point in the model must be equal to that at a corresponding point he prototype. Hence geometric and kinematic similarity demands that the ling velocities in the suspension be scaled in the same ratio as the fluid ocities.

The third type of similarity concerns the changes in settling velocities. pose that in the prototype, the mean settling velocity changes by an amount in some distance Δx. For similarity $\overline{w}$ must change in the model such the dimensionless rate of change is the same for both model and proto- e. Denoting scale model quantities by s and prototype quantities by p it ows that

$$\frac{\left(\frac{\Delta \overline{w}}{U_o}\right)_s}{\left(\frac{\Delta x}{x_o}\right)_s} = \frac{\left(\frac{\Delta \overline{w}}{U_o}\right)_p}{\left(\frac{\Delta x}{x_o}\right)_p}$$

$$\frac{\left(\frac{\Delta \overline{w}}{\Delta x}\right)_s}{\left(\frac{\Delta \overline{w}}{\Delta x}\right)_p} = \left(\frac{U_o}{x_o}\right)_s \left(\frac{x_o}{U_o}\right)_p \tag{13}$$

If, like most open channel models the model was designed according
Froude model law, Eq. (13) becomes

$$\frac{\left(\frac{\Delta \bar{w}}{\Delta x}\right)_s}{\left(\frac{\Delta \bar{w}}{\Delta x}\right)_p} = \left[\frac{(x_o)_s}{(x_o)_p}\right]^{-1/2}$$

The flocculation may be related to turbulent mixing which is scaled acc
to Froude's law as follows

$$\frac{(e_o)_s}{(e_o)_p} = \frac{(U_o\, x_o)_s}{(U_o\, x_o)_p} = \left[\frac{(x_o)_s}{(x_o)_p}\right]^{3/2}$$

Consequently the ratio of the effect of flocculation is equal to the negati
power of the length ratio while the ratio of a mechanism contributing to
lation is equal to the three-halves power of the scale ratio.

Eqs. (14) and (15) show that the response of the suspension to flow c
ditions cannot be the same in model and prototype. It follows that a sus
with "scaled" settling properties must be used in the model. Usually, h
ever, the same suspension must be used in model and prototype. In thi
ation there are two approaches to using models.

The first approach is to use the model to predict only the flow in the
type; separate experiments are performed to study the settling propert
the suspension. The measured settling properties are then used to calc
the removal that will occur in the predicted flow. This method is suite
study of such problems as silt deposition in reservoirs, sedimentation
mouths of rivers, and the diffusion and sedimentation of sewage on othe
wastes in bays and estuaries.

In the second approach the settling is studied in the scale model. Th
model results are then "scaled up" to predict prototype results. In ord
perform this scale-up, it is necessary to develop a scale equation givi
removal in the prototype as a function of removal in the model. From
foregoing analysis it is obvious that such an equation should be based a
on the properties of the suspension as on the model laws. Therefore s
experiments on the settling properties should be conducted in conjuncti
the model tests. This second approach is frequently used in pilot plant
of settling tanks.

3. Research on Quiescient Settling

a. Purpose and Scope

The purpose of the research described in this section was to develo
methods for determining the settling properties of an individual suspen
More specifically, the objective was to measure the settling velocity d
bution of a suspension, the local mean settling velocity at various poin

pension, and the factors affecting the local mean velocity. Most of the
k was confined to quiescent settling.

The approach was both experimental and theoretical. The experiments con-
ted of allowing a suspension to settle quiescently in a vertical tube. During
settling, small samples were withdrawn from various locations in the tube,
these were analyzed for suspended-solids concentration. To supplement
experiments, theoretical analyses were made of the quiescent settling of
crete particles in a settling tube, the kinetics of flocculation during quies-
t settling, and the analyses of data from the experiments.

Settling Velocity Distributions

The purpose of the first phase of the research was to measure the settling
ocity distribution of many varied suspensions. The pipette analysis[7] was
d for the measurement. A portion of a suspension was shaken and allowed
settle in a vertical glass tube. During settling, samples were withdrawn
m a known distance below the top surface of the suspension by means of a
ken-tip pipette or small glass tube.

The results of some of the experiments are shown in Figs. 2, 3 and 4. The
inate represents the ratio of the particle concentration in the sample to the
rage concentration of the portion at the beginning of settling, while the ab-
ssa indicates that the sample was taken at depth z below the surface at time
ter the beginning of settling.

Under the following conditions, a curve from these figures represents a
nulative frequency distribution of settling velocities.

(1) At the beginning of settling the particles of each class of settling veloci-
ties were uniformly distributed throughout the suspension.
(2) No flocculation, turbulence, or hindered settling occurred during the
experiment. In this case, the settling velocity distribution is the major
settling property of the suspension and is often sufficient for calculating
removal. Dobbins[3] and Camp[13,16] have calculated the removal for

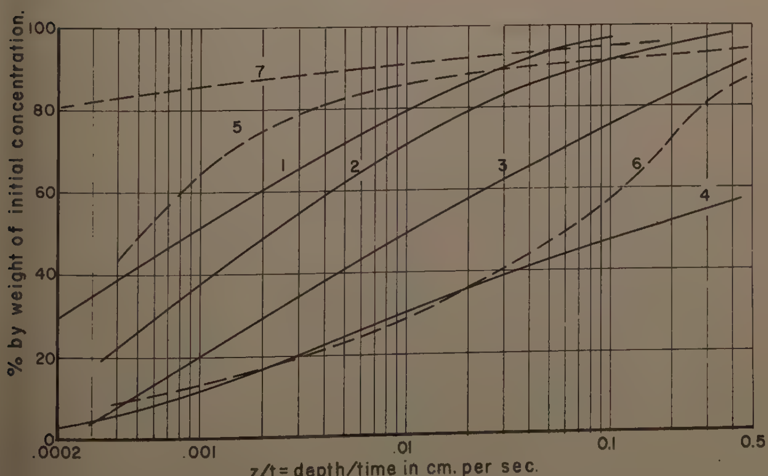

Fig. 2 Comparison of pipette analyses (see table 1)

certain cases of particles settling without flocculation or hindere
ling.

Fig. 2 shows the results of pipette analyses performed on primary e
from two large sewage treatment plants, primary and secondary effluen
a third, and various mixtures of effluent with digested sludge and sea w
with digested sludge. The analyses were made by the writer in connecti
with studies of the marine disposal of sewage and sludge in the vicinity
Angeles, California.(8) These were made in an apparatus which maintai
the temperature of the suspension at 31.5° C. The average depth of sam
was about 40 cms for each analysis.

Even though the pipette analysis takes no account of flocculation or h
settling, the curves of Fig. 2 indicate large differences between the sett
properties of the various suspensions. For example, curves 1, 2 and 3
represent primary effluents from sewage treatment plants. However, t
proximate median settling velocities of suspensions 1 and 3 differ by a
of 10.

Pipette analyses were also performed on raw Pasadena sewage obtai
from a trunk sewer of the Los Angeles County Sanitation Districts. Thi

TABLE 1

Sources of the Suspensions Represented in Figure 2

Curve	Source	Initial Concentra (equal to Susp. So
1	Primary effluent from the treatment plants of the Orange County (California) Sanitation District	893 mg/1
2	Primary effluent from the Hyperion Sewage Treatment Plant (City of Los Angeles).	212 mg/1
3	Primary effluent from the Joint Disposal Plant of the Los Angeles County Sanitation Districts	314 mg/1
4	Primary effluent plus 1% by volume of digested sludge from the Joint Disposal Plant, LACSD	527 mg/1
5	Primary effluent, plus elutriation effluent from Hyperion Sewage Treatment Plant	289 mg/1
6	One part digested sludge from Hyperion Sewage Treatment Plant plus 19 parts sea water	32,200 mg/1
7	Secondary effluent from Hyperion Sewage Treatment Plant	33 mg/1

er serves most of Pasadena, San Marino, South Pasadena, and parts of tiguous communities, with a total sewered population of about 200,000. The ples obtained from the sewer were gross samples; they were taken in the rning and tested in the afternoon of the same day. Before the experiment, gross sample was poured into a large ceramic crock. When a smaller tion was required for testing, the gross sample was stirred, and while ring continued, the smaller portion was taken.

A five-gallon gross sample was obtained on each of several days, and a tion was taken from each for pipette analysis. General information about analyses is given in Table 2 and the results are plotted in Fig. 3. These ves show a variation in settling properties of the sewage from one run to next. For some engineering purposes, it would be desirable to represent data from all four runs by a single curve. The median settling velocity such a curve would be about 0.02 cm. per sec., and the median velocities the individual curves differ from this value by a factor of 1.5 or less. s deviation is small compared to the differences between median velocities Fig. 2. Hence, for the purpose of comparing Pasadena sewage with the sussions of Fig. 2, all four runs could be represented by a single curve.

From the curves of Figs. 2 and 3, two conclusions can be drawn. First, pension which may be considered as similar can have significantly differsettling properties. Second, even for a suspension as heterogeneous as age, the difference between runs for a single suspension can be small pared to the difference between suspensions.

Table 2

Pipette Analyses of Pasadena Sewage

ttling tube - 1 liter graduate

lume used - 1 liter

mperature control - none

ze of samples withdrawn - 25 ml.

ethod of withdrawing samples - 25 ml. broken tip pipette lowered into suspension by hand for each sample.

pth of samples - 22 cm. below surface of sewage.

Date	Gross Sample Obtained	Pipette Analysis Began	Initial Concentration	Temp. During Test
ur. Oct. 21/54	8:30 am	2:35 pm	304 mg./ℓ	
ur. Oct. 28/54	8:45 am	11:40 am	320	22-27°C.
n. Nov. 15/54	9:00 am	1:45 pm	374	21-23.5°C
n. Nov. 22/54	9:00 am	1:00 pm	380	23-30°C

Part of the difference between the curves of Fig. 3 may be due to sa error. Whenever a portion is withdrawn from a heterogeneous suspens such as raw sewage, the properties of the portion may not be the averag properties of the suspension. To test this error, tests were made on tv portions from a single gross sample. By means of two identical glass tubes placed in a single constant-temperature water bath, two similar p of sewage from a single gross sample were subjected to identical pipett analyses. The portions were approximately four liters in volume.

The experiment was rerun for a second gross sample, and the result both runs were plotted on the graph shown in Fig. 4. The difference bet curves for one run is nearly as large as the difference between curves in Fig. 3. Hence, some of the variation in the latter curves is due to sa errors.

Sampling errors can be reduced by increasing the volume of the port used in the suspension. The portions used in these pipette analyses va tween 1 and 4 liters, and they appear to be too small. Hence larger sa are recommended for suspensions of considerable heterogeneity.

c. Factors Affecting $\overline{W}$

Unless the particles of a suspension settle without flocculation or hi settling, the pipette analysis gives only a rough indication of the settlin properties of a suspension. In a more general situation it is necessary an experiment that will show how flocculation and hindered settling affe settling velocities of the particles.

In hindered settling the concentration of suspended particles is high for the presence of one particle to affect the settling of its neighbors. flocculation, on the other hand, a faster particle overtakes a slower on becomes attached to it. The two settle henceforth as a unit with a velo usually different from the original velocity of either particle. These tv fects cannot be separated in an experiment. If they occur simultaneous result is a single change in $\overline{W}$. However, in order to find out precisely

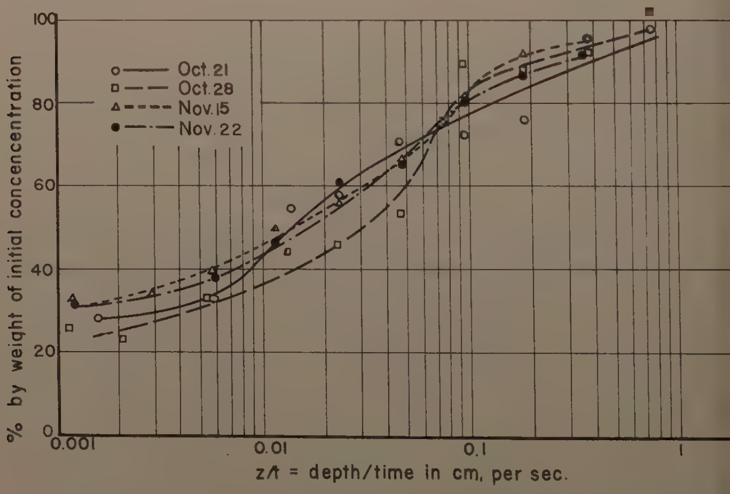

Fig. 3. Pipette analyses of Pasadena sewage (See Table 2).

y produce the changes in $\overline{w}$ it is convenient to think of them separately.
The hindered settling of uniform particles without flocculation has been died theoretically and experimentally by McNown and Lin[9] and inhour,[10] among others. The former found that the individual settling ocity of a particle depends upon the volume concentration of the particles l upon the Reynolds number of the particle. Steinhour, on the other hand, oduced a formula giving velocity as a function of volume concentration alone. is formula was based on experiments where particles were settling in the kes range.

Both of these investigators considered cases where the particle concention was uniform throughout the settling tube. Kynch[11] went further and de a theoretical analysis of uniform particles in a tube where the concention varied with depth. By assuming that the individual particle velocity ended solely on the concentration in the neighborhood of the particle, he ained results which agree with experimental observations on the subsidence hick slurries.

On the basis of these studies it was concluded that the ratio of the settling ocity at concentration ϕ to velocity at $\phi = 0$ depends primarily on ϕ, with ticle velocity as a secondary factor. Hence, in a suspension with particles many velocities, a change in ϕ will cause the velocity of each particle to nge by the same ratio. However, the magnitude of the change in velocity l equal the product of the ratio and the particle velocity. Thus, the magnie will increase with particle velocity.

It follows, that for a given change in ϕ the mean settling velocity will nge by an amount depending upon the distribution of settling velocities at time of the change. This distribution can probably be characterized by the an settling velocity and the standard deviation, σ, of the velocities. Theree, changes in $\overline{w}$ will be a function of $\overline{w}$ itself, σ and the volume concention of particles.

For flocculation no theories existed which were suitable for this research. previous theories were all confined to predicting the rate of interparticle tacts, while ignoring the effect on settling velocities. Furthermore, they

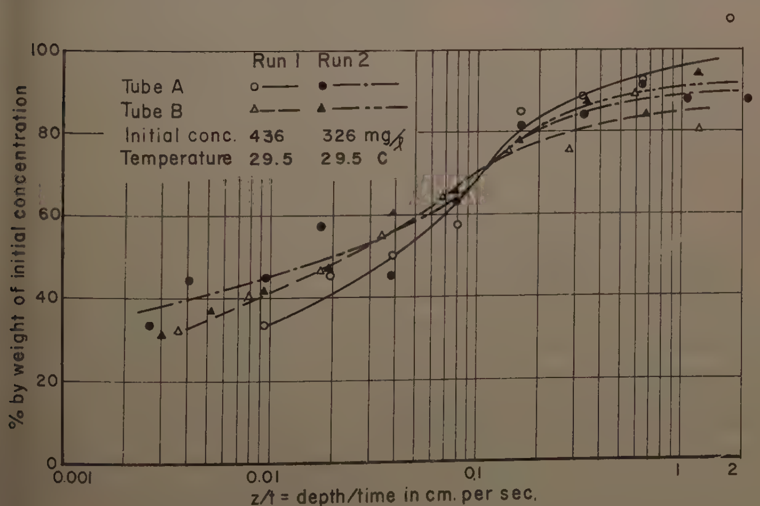

Fig. 4. Duplicate pipette analysis Pasadena sewage.

dealt with suspensions having velocities of only a few classes. Therefo
kinetics of flocculation during quiescent settling was studied theoretical
for the purpose of discovering how flocculation produces changes in $\overline{w}$.
cause of the number of unknowns the theory could not be completed, but
carried far enough to show that the change in $\overline{w}$ at a point depends on th
following properties of the suspension at the same point.

(1) The volume concentration of particles.
(2) The mean settling velocity of particles.
(3) The standard deviation of the settling velocities of the particles.
(4) The fraction of inter-particle contacts that result in the union of particles.

Item (4) is simply a means of accounting for the surface chemistry of t
particles.

The value of $\overline{w}$ at a point will also change in free settling. That is,
absence of flocculation and hindered settling it is easily shown(12) that

$$\frac{\partial(\overline{w}\,\phi)}{\partial t} + \frac{\partial}{\partial z}\left(\sigma^2 \phi + \overline{w}^2 \phi\right) = 0$$

Thus, the change in $\overline{w}$ depends on $\overline{w}$ itself and on the standard deviatio
settling velocities.

It appears, therefore, that the first three of the items listed above i
nection with flocculation are also the most important items in connecti
free and hindered settling. Consequently, an experimental study of qui
settling should involve the determination of at least $\overline{w}$, σ, and concentr
volume. The determination of the first of these, $\overline{w}$, is described in par
which follows.

d. Measurement of $\overline{w}$

Since a measurement of $\overline{w}$ has not been reported in the literature, s
time was spent in devising a method. Finally, it was decided to use an
periment based on a suggestion by Camp.(13) The suspension was allo
settle quiescently in a vertical tube as in a pipette analysis. During th
ling a series of samples were taken at each of several depths, and thes
ples were analyzed for particle concentration. This experiment was gi
name multiple-depth pipette analysis or, simply multiple-depth analysi
Some very simple equations will show how such an analysis can be use
measure $\overline{w}$.

For the purpose of discussion, consider the hypothetical settling tub
on the left in Fig. 5. All horizontal cross-sections of the tube are of u
and at any time, the particle concentration, ϕ, over any such cross-se
constant. Hence, ϕ is a function of z, the depth below the top of the su
and t, the time after beginning of settling.

At the beginning of the test ($t = 0$) pipette samples are withdrawn si
taneously at many depths. The results are plotted as the curve, $t = 0$,
diagram on the right of Fig. 5. At $t = T_1$, another set of samples is wi
and the results are plotted as the curve, $t = T_1$. The curves, $t = T_2$ an
represent similar operations at times T_2 and T_3, respectively.

Physically speaking, each of these curves is a profile of the concen
at some given instant. When all the profiles from an analysis are put
diagram, it can be called the concentration profile diagram. Mathema

aking, this diagram is a plot of $\phi(z,t)$ as a function of z with t as a parame-. At any stage in the discussion, it is possible to give the diagram a physi- or mathematical interpretation, depending on which is more illuminating.

The concentration profile diagram is used in conjunction with the continuity ation for the calculation of $\overline{w}(z,t)$. First, it is noted that for the settling e, Eq. (8) is reduced to the following:

$$\frac{\partial\phi}{\partial t} + \frac{\partial(\overline{w}\phi)}{\partial z} = 0 \tag{17}$$

egrating this equation with respect to z gives

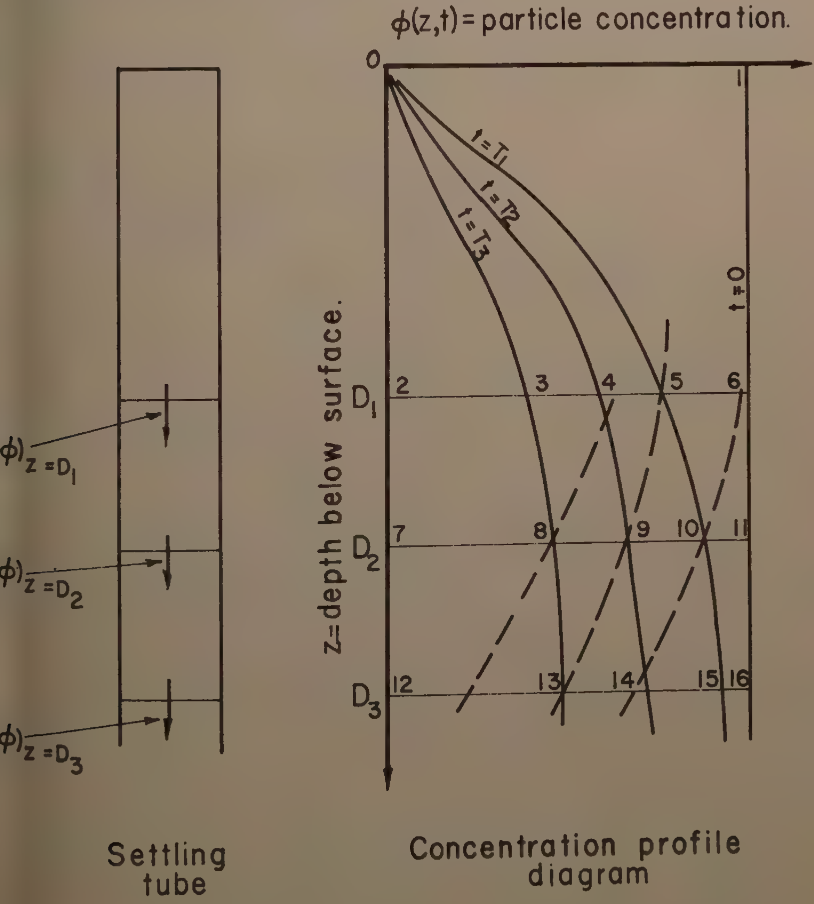

Fig. 5. Analysis of data from multiple-depth pipette analysis.

$$(\overline{w}\,\phi)_{z=D} = -\int_0^D \frac{\partial \phi}{\partial t}\,dz$$

$$= -\frac{\partial}{\partial t}\int_0^D \phi\,dz$$

Eq. (18) shows how $\overline{w}$ can be calculated. A value of D is selected; f ample, consider D_1 in Fig. 5. The areas 025, 024, and 023 are calcula the values are plotted against t. If this is done for a sufficient number files, the result will be a smooth curve giving the area under the ϕ pro above D_1 as a function of time. The slope of the curve is precisely the hand side of Eq. (18), and hence is equal to $\overline{w}\phi$ at $z = D_1$. The process repeated for any value of D.

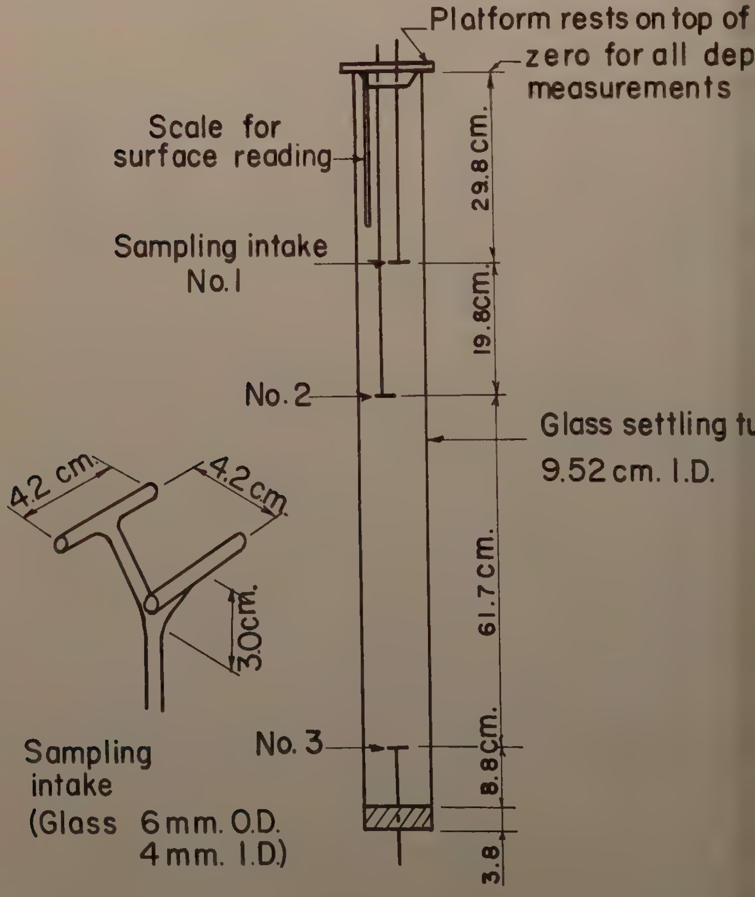

Fig. 6. Multiple-depth settling tube.

In order to test the feasibility of this approach to measuring $\overline{w}$, a pilot ex-iment was performed using the apparatus shown in Fig. 6. Sampling in-es, or pipettes, were located at three depths as shown. Samples could be wn through the top two by means of a vacuum and through the bottom one means of gravity flow.

The suspension to be tested was thoroughly mixed in a separate container then poured quickly into the tube. Since the upper two intakes are attached platform which rests freely on the tube, these intakes could be put in ce immediately after pouring. After they were in place, the elevation of the surface was read on the scale shown and the withdrawing of samples began. The surface elevation was read again after the withdrawal of each sample. h these readings it was possible to allow for the lowering of the surface in imating the distance settled by particles. The estimated distance was used z for each sample. Time, t, was measured from the end of pouring.

Table 3

Multiple-Depth Analysis of a Suspension of

Clay and Alum in Water

Time After Start of Settling	Intake	Depth of Intake z	z/t	Volume of Sample	Temp. of Sample	Weight of Particles in Sample	% of Initial Conc.
sec.	No.	cm	cm/sec	ml.	°C	mg	
0					29.5*		
60	2	42.6	0.71	28		17.7	99
90	3	104.3	1.2	27		17.3	104
120	1	21.7	0.18	32 1/2		20.3	99
180	3	103.2	0.57	33		21.6	104
245	2	39.7	0.16	28		18.1	103
300	1	18.4	0.062	37		23.6	102
360	3	101.4	0.28	31 1/2		20.0	101
480	1	17.1	0.036	28		16.7	95
600	2	37.5	0.063	31 1/2	30.3	21.4	108
720	3	99.2	0.14	32	30.3	22.9	114
960	1	15.2	0.016	22		8.1	59
1200	2	35.6	0.030	25		11.2	71
1440	3	97.3	0.068	36		12.9	59
1920	1	13.2	0.069	27		7.7	45
2400	2	33.7	0.014	31		9.0	46
2880	3	95.4	0.033	29	31.5	7.2	40
3840	1	11.1	0.0029	31	32.0	5.1	26
4860	2	31.6	0.0065	32	32.3	5.0	25
5760	3	93.3	0.016	27	32.8	4.6	27
7740	1	8.9	0.0012	31	33.4	2.7	14
10100	2	29.4	0.0029	40	34.0	4.1	16
14100	3	91.1	0.0065	26	34.8	1.2	7

* Measured in mixer

Initial Concentration = 655 mg/l

Alum Concentration = 25 mg./l

This apparatus was used for the multiple depth analysis of a suspen
bentonite clay in Pasadena tap water and alum. $(Al_2(SO_4)_3 \cdot 18\ H_2O)$.
initial concentration of clay was 655 mg. per liter, while that of alum v
mg. per liter. The results of the experiment are given in Table 3 and
in the concentration profile diagram of Fig. 7. Since samples were tak
only three depths some intermediate plotting and interpolation were ne
to produce the profiles.

Once the profiles were obtained, the mean settling velocity $\overline{w}(z,t)$ w
calculated in the manner described above. Values for three values of z
given in Fig. 8. It is possible to calculate the order of magnitude of

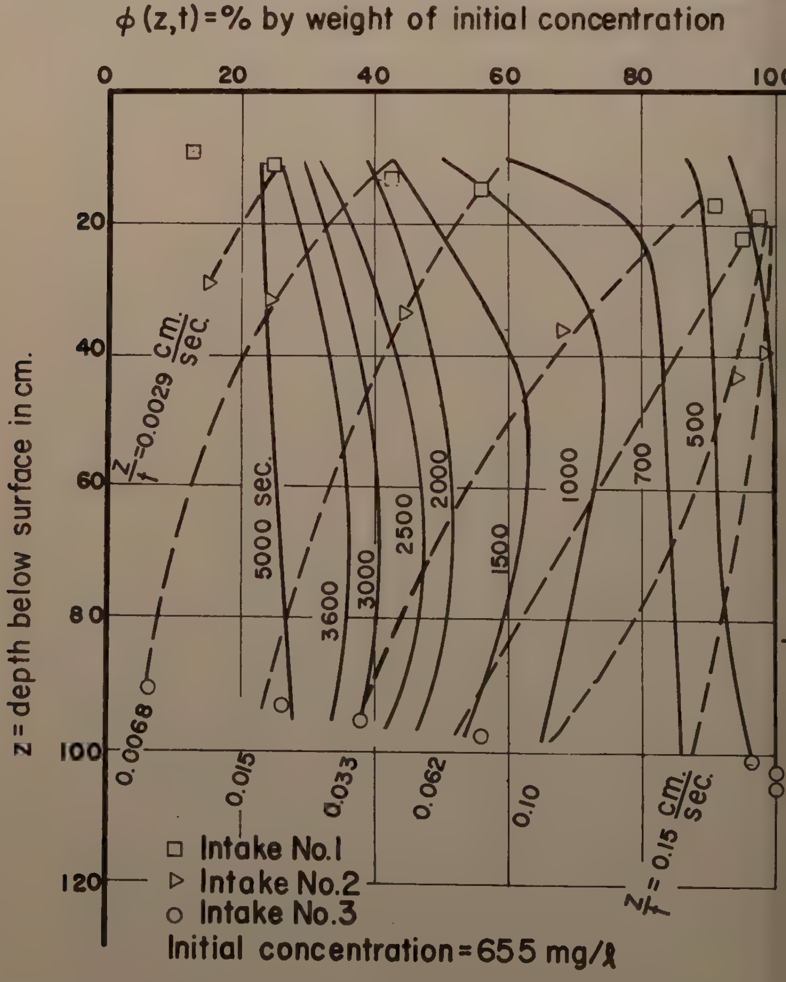

Fig. 7. Concentration profile diagram, Bentonite
clay and alum in water.

:culation effects on local settling. At a depth of 90 cm. the mean settling ocity increases from 0.021 cm. per second at t = 200 seconds to 0.037 at 500 seconds. This change represents an increase of 75 per cent and an erage rate of increase of 0.000053 cm. per sec. per sec. Furthermore, the nge in concentration during the same time was only 3 per cent. The effect particles settling out of suspension must have been small. Therefore, the e of change is primarily due to flocculation.

It is possible to tell directly that flocculation is causing the particles to eed up by looking at the dashed lines constant z/t in Fig. 7. The physical nificance of these lines is best explained in the following manner. An ob-ver starts at the surface of the suspension at t = 0 and descends through suspension at a constant velocity. The concentration that he observes at ious depths is given by a line of z/t equal to the velocity.

It can be shown[12] that when neither hindered settling nor flocculation oc-, these lines are straight and parallel to the z-axis. If hindered settling ws the particles down more than flocculation speeds them up, the lines pe away from the z-axis as depth increases. If the converse is true, the es slope toward the z-axis so depth increases. For Fig. 7, then, it is seen t flocculation is the predominant effect.

The maximum mean settling velocity for z = 90 cm. occurs at 700 seconds. value of 0.056 cm. per second is 2.8 times the mean settling velocity at 200 seconds. The average rate of change between t = 200 seconds and 700 seconds was 0.00007 cm. per sec. per sec.

A maximum value of $\overline{w}$ occurs at all depths in the settling tube. After this ie, the mean settling velocity decreases. Particles are settling out of sus-sion, and the loss of faster particles offsets the effect of flocculation.

While this work was being done, Fitch[14] presented the results of a ltiple-depth experiment on a suspension of whiting ($CaCO_3$) and ferrisul $_2(SO_4)_3 \cdot 9H_2O)]$ in water. The concentration of $CaCO_3$ was 400 parts per lion (ppm), while the concentration of ferrisul was 15 ppm. This suspension

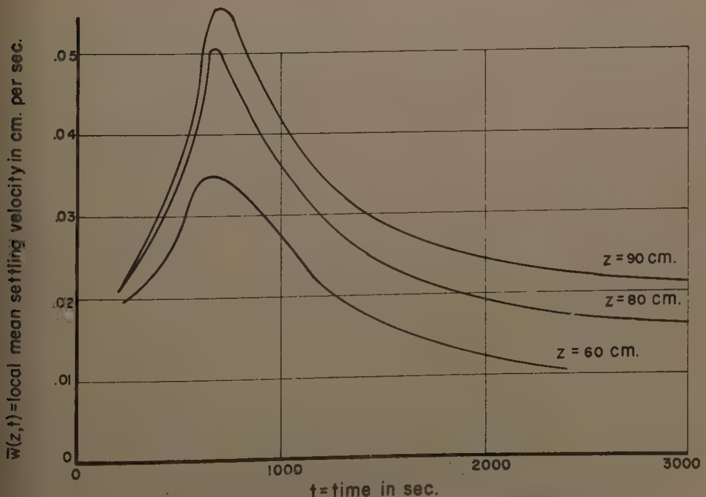

Fig. 8. Local mean settling velocity as a function of time. Bentonite clay and alum in water.

was allowed to settle in a tube seven feet deep and 5-1/2 inches in inte
diameter. During the settling, samples were taken at seven depths by
of veterinary hypodermic needles which passed through the walls of th
For temperature control, the outside of the tube was covered with insu
one inch thick.

Fitch's results are presented as a concentration profile diagram in
The solid curves are concentration profiles or lines of constant time t

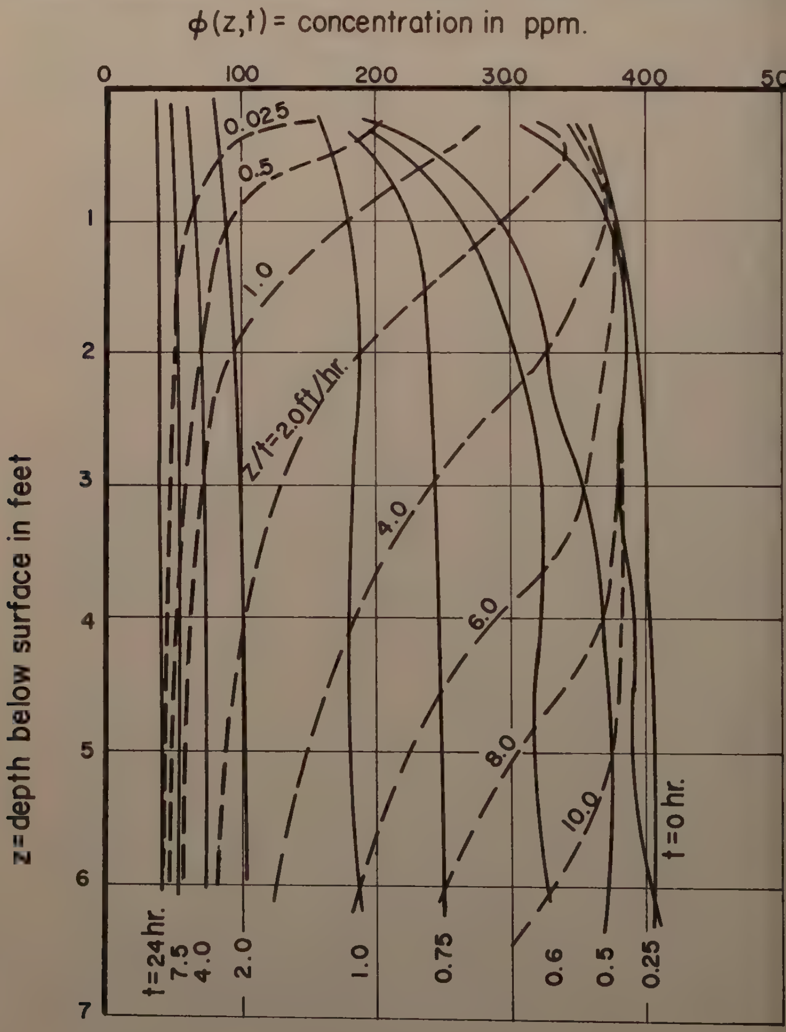

Fig. 9. Concentration profile didgram, $CaCO_3$ and $Fe_2(SO_4)_3 \cdot 9H_2O$ in water (Based on data by Fitch)

lashed curves are curves of constant z/t. For $t > 0.25$ hours and $z > 2$ the constant z/t lines slope toward the z-axis, showing the effect of ulation.

ollowing the method described above, the local mean settling velocity was ilated. The mean velocities for z = 3 feet and z = 6 feet are shown in 10. At z = 6 feet, the mean settling velocity increased from 0.013 cm. sec. at t = 0.25 hours to 0.035 cm. per sec. at t = 0.5 hours. The average of change during this time was 0.000025 cm. per sec. per sec. At t = 0.5 s, the concentration at z = 6 feet is 94 per cent of the initial concentration. equently, up to this time the change in $\overline{w}$ is caused primarily by floccu- n.

is interesting to compare the change in $\overline{w}$ for whiting and ferrisul with hange for clay and alum. The comparison is valid only for those stages ttling for which the concentration has not decreased significantly. For ng and ferrisul, the rate of change during this stage of settling was 0025 cm. per sec. per sec. For clay and alum, the corresponding rate of ge was 0.000053 cm. per sec. per sec. The values differ by a factor of

ne important practical conclusion can be drawn from these determinations That is that the effect of flocculation increases with depth from the ice. This is shown by the curves for $\overline{w}$ in Figs. 8 and 10. If the settling in either case had been deeper, the peak values of $\overline{w}$ would probably have eased with depth until a limiting value was reached.

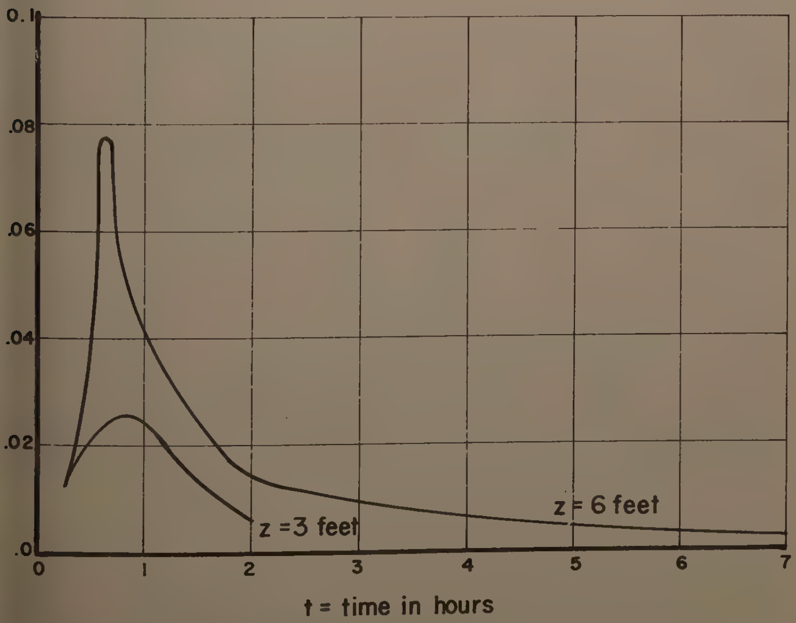

Fig. 10. Local mean settling velocity as a function of time (based on data by Fitch).

e. Proposals for Future Work

The next logical step in this work is to design an experiment in wh possible to measure $\overline{w}$, σ, and particle concentration. For such an ex ment, it is desirable to have a tube about ten feet deep with 7 or 8 sa intakes. The sampling intakes should enter through the walls of the t to cause a minimum obstruction to settling particles. The tube shoul housed in a water jacket for temperature control, but both jacket and should have windows for visual observations.

The cross-sectional area of this tube should be at least equal to th circle twelve inches in diameter. Because of this large area, the sa withdrawn during settling can be as large as 500 ml. Then each samp be allowed to settle in a 500 ml. graduate for a simple pipette analysi

Five or six 25 ml. samples from each 500 ml. graduate should be give a settling velocity distribution for each 500 ml. sample. From t distribution it is possible to calculate the mean and standard deviatio settling velocities for each sample. Of course, flocculation may occu graduate, but the effect is small compared to that in the tube.

The same 500 ml. samples can be used for a determination of ϕ a concentration by volume. From the former the concentration profile drawn. The values of $\overline{w}$ can then be calculated as shown above and c against the value obtained from the simple pipette analysis.

Before this more elaborate multiple depth-analysis could be perfo the experiments on settling analysis were discontinued, and, as yet, r vision has been made for their continuance. It is hoped that someone the problem of sufficient interest to continue the work. If so, it will nomical to modify the tube to allow for the study of turbulence.

It is a simple matter to extend the proposed multiple-depth analys clude effects of turbulence. Rouse(15) and Dobbins(3) have shown tha form field of turbulence can be created by placing a vibrating grid in settling tube. Furthermore, Dobbins devised a method for controllin of particle resuspension at the bottom of the tube. If these additions corporated into the apparatus proposed above, it will be possible to s flocculation in a turbulent fluid.

4. The Multiple-Depth Analysis in Approximate Calculations of Re

a. Short-Cut Methods in Calculating Removal

There are times when the flow in a channel is such that V = W = 0 independent of depth. Then, the settling of particles is similar to se a vertical tube which is moving at a velocity U in the x-direction. T files in the tube at time, t, corresponds to profiles in the channel at x. It follows that these profiles can be used directly to calculate re

b. "Ideal" Settling Tanks

Part (a) of Fig. 11 shows the settling zone of a rectangular settli length L, width B, and depth D. By making certain simplifying assu about such a basin, Camp(13) devised the concept of an "ideal tank". the nomenclature of Section 2, these assumptions can be stated as fo

$$U(x,y,z) = \text{constant}$$
$$V(x,y,z) = W(x,y,z) = 0$$

$e_x(x,y,z) = e_y(x,y,z) = e_z(x,y,z) = 0$
$f_i(o,y,z) \;=$ constant for each value of i

No resuspension at the bottom of the tank

Although these assumptions are sufficient to describe the tank, Camp also umed that all particles in the tank settle without flocculation or hindrance. ce this last assumption is unnecessary, in this paper the term ideal tank l refer only to the assumptions about the tank. In place of the assumption

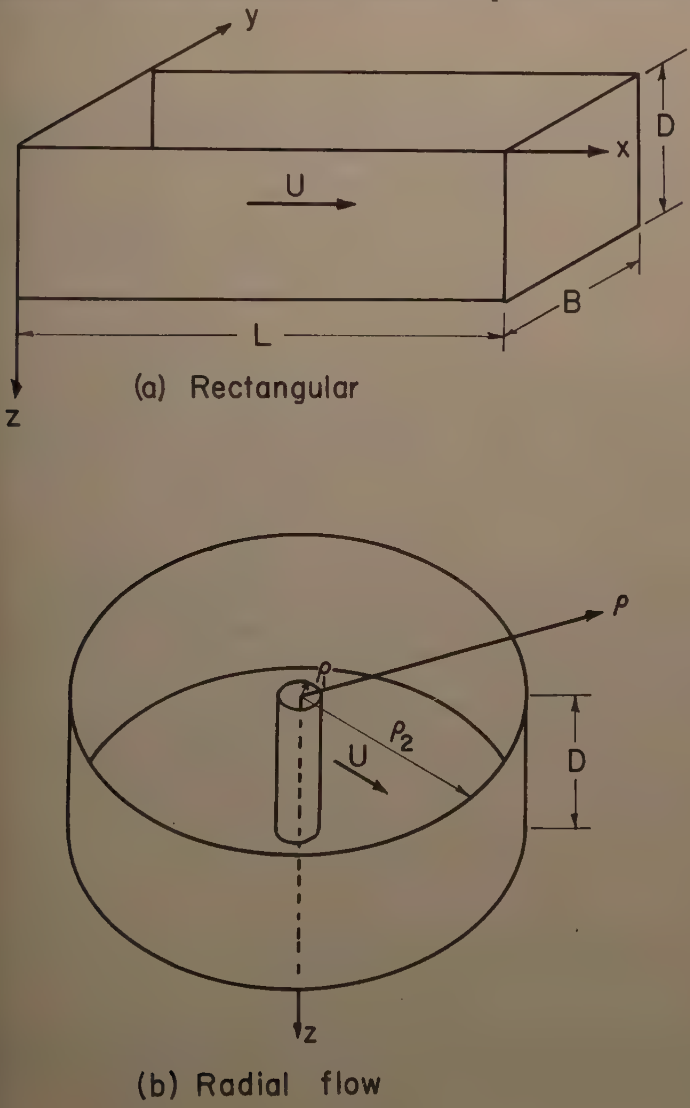

Fig.11. Settling zones of ideal tanks.

about the settling, the results of a multiple-depth analysis can be used. approach is based on the fact that in an ideal tank settling is identical in a vertical tube, and concentration profiles at distance x in the tank c spond to profiles at time t in the tube. If the concentration profiles fro tube are used, the removal in the ideal tank will be given by

$$R(x) = B\,U \int_{o}^{D} \left[\phi(z,o) - \phi(z,t)\right] dz$$

where

$$t = \frac{x}{U}$$

and $\phi(z,t)$ represents concentration in the tube.

In settling tanks, the removal ratio, r(x), is more important than th moval itself. The former is defined as the fraction of particles enteri tank that settle out in distance x. Hence,

$$r(x) = \frac{R(x)}{\int_{o}^{D} \phi(z,o)\, dz} = \frac{\int_{o}^{D} \left[\phi(z,0) - \phi(z,t)\right] dz}{\int_{o}^{D} \phi(z,0)\, dz}$$

The integrals in Eqs. (19) and (20) can be represented on the concentr profile diagram by areas. Consider, for example, the diagram shown 5. For a tank of depth D_1, and a distance of UT_1, the removal is repr by area 0561 while the removal ratio is represented by

$$r(x) = \frac{\text{area } 0561}{\text{area } 0261}$$

A similar approach can be used for radial flow tank. Part (b) of F shows what might be considered an "ideal radial flow tank". Fluid en tank through the surface of a cylinder of radius ρ_1, centered along the axis of the tank. It then flows radially and horizontally to the cylindri of the tank. These walls have a radius of ρ_2.

The flow conditions for this tank are assumed to be

$$U = \frac{b}{\rho} \qquad (b = \text{a constant})$$

$$V = W = 0$$

$$e_x = e_y = e_z = 0$$

The initial conditions are assumed to be

$$f_i = \text{constant at } \rho = \rho_1 \text{ for each value of } i,$$

while the bottom boundary condition is the same as that for Camp's ic

Settling in this radial tank is identical to settling in a vertical tube Concentration profiles at radius ρ in the tank correspond to profiles :

$$t = \frac{\rho^2 - \rho_1^2}{2b}$$

the tube. It follows that removal in the tank in radial distance, ρ, is given

$$R(\rho) = 2\pi \rho_1 \int_0^D U(\rho_1)\, \phi(\rho_1, z)\, dz - 2\pi\rho \int_0^D U(\rho)\, \phi(\rho, z)\, dz$$

$$= 2\pi b \int_0^D \left[\phi(z, o) - \phi(z, t)\right] dz \qquad (22)$$

ere t is given by Eq. (21). Eq. (22) shows that removal in the radial tank is presented by an area on the concentration profile diagram just as it is for e rectangular tank.

It is suggested that the combination of assuming an ideal tank and perform- g a multiple-depth analysis on the suspension will give a first approximation the behavior of the tank. This approximation will often be better than mak- g an elaborate study of the hydraulics of the tank while ignoring the proper- s of the suspension.

Ideal Tanks with Turbulence

For a better approximation to a rectangular or cylindrical basin, it is ssible to add turbulence to the ideal tank. If the turbulent diffusion is as- med to be

$$e_x = e_y = 0$$

$$e_z = \text{constant},$$

e settling in the tank will be equivalent to settling in a tube into which uni- rm turbulence is introduced.

Dobbins(3) and Camp(16) have assumed this sort of tank in a study of the ect of turbulence in retarding settling. Camp assumed that the particles ttled without flocculation or hindrance. However, this assumption is un- cessary, since it is a simple matter to put the actual suspension in a settling be as deep as the tank and introduce turbulence by means of a vibrating grid.

The combination of assuming an ideal tank and performing a turbulent ıltiple-depth analysis on the suspension is suggested as a second approxi- ıtion to the behavior in a rectangular basin. For better approximations, it necessary to obtain enough information for the step calculation outlined in ction 2.

Overflow Rate and Detention Time in Ideal Tanks

In the technical literature, one finds a great deal of discussion about ether a settling tank should be designed on the basis of overflow rate or de- ıtion time. The discussion about the merits of these two approaches is in- nclusive, because it disregards the properties of the suspension. It will w be demonstrated that whether overflow rate or detention time should be ed depends upon the nature of the suspension. Indeed, for a single tank, the moval ratio may be closely related to overflow rate for one suspension, to ention time for a second, and to neither for a third.

The detention is simply the average time that an element of fluid re in the tank. Let it be called T. For the rectangular ideal tank,

$$T = \frac{L}{U}$$

while for the radial,

$$T = \frac{\rho_2^2 - \rho_1^2}{2b}$$

Overflow rate, on the other hand, is obtained by dividing the volume flow through the tank by the horizontal area of the tank. Let it be w_o. rectangular basin,

$$w_o = \frac{UBD}{LB} = \frac{D}{T}$$

while for the radial,

$$w_o = \frac{2\pi \rho_2 \frac{b}{\rho_2} D}{\pi(\rho_2^2 - \rho_1^2)} = \frac{D}{T}$$

Eqs. (25) and (26) show that for constant flow rate and overflow rat tention time varies with depth. Thus, if the effectiveness of the tank i pend on overflow rate, r(x) will not vary with D as long as D/T is cons Conversely, if the effectiveness depends on detention time, r(x) will nc with D as long as T is constant.

To study the problem experimentally, a multiple-depth analysis is formed on the suspension concerned. The settling tube for this analys should be as deep as the deepest possible tank to be considered. The from the analysis are plotted on a concentration profile diagram as sh Fig. 5.

In this figure, let D_1, D_2 and D_3 represent the depths of three tank considered. The detention times for these tanks are T_1, T_2 and T_3, r spectively. The tanks all have the same overflow rate. Therefore,

$$\frac{D_1}{T_1} = \frac{D_2}{T_2} = \frac{D_3}{T_3}$$

and the dashed line 5, 9, 13 is a line of constant z/t. The other dashe are also lines of constant z/t.

If the removal ratio is to depend on overflow rate alone, then

$$\frac{\text{area } 0561}{\text{area } 0261} = \frac{\text{area } 0, 9, 11, 1}{\text{area } 0, 7, 11, 1} = \frac{\text{area } 0, 13, 16, 1}{\text{area } 0, 12, 16, 1}$$

Furthermore, this type of relationship must hold for any overflow rat for any dashed curve of constant z/t. It can be shown(12) that Eq. (27 hold for arbitrary overflow rate if and only if the lines of constant z/t straight and parallel to the z-axis.

If, on the other hand, removal ratio depends only on detention time equation of the form

$$\frac{\text{area } 0, 5, 6, 1}{\text{area } 0, 2, 6, 1} = \frac{\text{area } 0, 10, 11, 1}{\text{area } 0, 7, 11, 1} = \frac{\text{area } 0, 15, 16, 1}{\text{area } 0, 12, 16, 1}$$

st hold for each profile. Eq. (28) will hold if and only if the profiles are straight lines parallel to the z-axis.

It follows that for quiescent settling in ideal tanks the problem of overflow e and detention time is determined by the pattern of the profile diagram. s pattern, in turn, is determined by the settling properties of the sus- sion.

As an example of a suspension for which detention time is more important, sider the data obtained by Fitch and plotted in Fig. 9 above. The profiles almost straight and vertical. Thus, for any tank up to six feet in depth, removal is affected more by detention time than by overflow rate. Calcu- ons by Fitch substantiate this conclusion.

With only these curves at hand, one might ask the following question. What l be the situation when this suspension settles in a rectangular tank ten t deep? To answer this question, it is necessary to try to sketch in the es of constant z/t between the depths of six and ten feet and draw the pro- s from these. After a few trials, it becomes evident that the profiles are likely to change their shape drastically. Hence, removal will still depend marily on detention time.

Detention Time and Overflow Ratio—General

The data by Fitch shows that when flocculation occurs in an ideal tank the noval ratio can be more closely related to detention time. Furthermore, in erence (12) it is shown that flocculation has a tendency to cause the verti- straight profiles which indicate the dependence on detention time.

The reason is quite simple. When particles settle without flocculation, the centration at a given time will normally increase with depth. Hence, the file slopes away from the z-axis. With flocculation, however, the faster ticles settling through the slower ones gather up the slower ones and drag m out of suspension. This tends to cause the concentration at any given e to decrease with depth. The combined result can cause the concentration be independent of depth, i.e., a straight profile parallel to the z-axis.

However, Camp[13] has shown that when the particles settle without floccu- on or hindered settling, removal depends on overflow rate alone. Between suspensions discussed by Camp and those discussed by Fitch are sus- sions for which both overflow rate and detention time are significant. The gram of Fig. 7 represents an example of this last situation.

When turbulence during settling is to be considered, the studies of overflow e and detention time should be based on results from turbulent settling lysis. As yet, the available experimental results of this type are too mea- to permit any conclusions.

Sediment Deposition in Canals

In the design of canals, it is necessary to consider minimum velocities for -silting. If the latter cannot be maintained, it is desirable to know where he canal the silt will be deposited. Some useful information about this blem can be obtained from a turbulent multiple-depth analysis.

This analysis should be performed in a tube as deep as the canal. Inside tube, the level and vertical distribution of turbulence should be as close as sible to that in the canal. The vertical distribution of turbulence can be ained by varying the grid spacing and bar size along the vibrating grid. e turbulence level can be obtained by adjusting the frequency of the rations.

The tube can be used to find out what velocity is necessary for kee particles in suspension. By letting the suspension settle in the tube v ous grid frequencies, it will be possible to decide upon the turbulence that keeps deposition to a reasonably small amount. The velocity in t which produces the turbulence is the necessary velocity in the canal.

For each grid frequency, the test should be run for a considerable After only a short time in the tube, the particles of a suspension may reasonably well suspended at a given level of turbulence. However, i lation occurs, the particles may increase the settling velocities and b settle out. This process may not be evident for thirty minutes or an

If a minimum velocity cannot be maintained in the canal, the resul analysis will show the pattern of silt deposition. For if the flow in th is fairly uniform, profiles at distance x in the canal correspond to pr the tube. Removal is calculated in the manner described for the idea

When the level of turbulence in the canal is sufficiently low, the re a quiescent multiple-depth analysis may give some information about sition. Fig. 12 represents the results of a quiescent analysis of bento

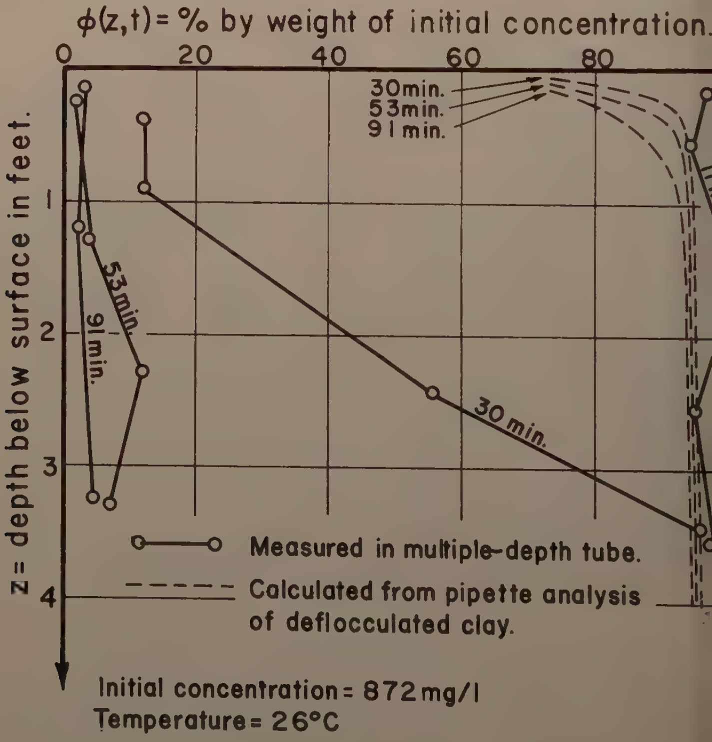

Fig. 12. Concentration profiles for bentonite clay in Pasadena tap water.

n Pasadena tap water. The analysis was performed in a lucite tube with an nternal diameter of 3-3/4 inches and a depth of 4 feet. Thus the settling night correspond, roughly, to settling during tranquil flow in an irrigation anal 4 feet deep.

In preparing the suspension, the clay was blended thoroughly with 0.5 liters f water. This blended mixture was then shaken with enough water to bring he final clay concentration to 872 mg per liter. After two minutes of vigorous haking the suspension was poured into the tube. This mixing might be simi- ar to that which the suspension would receive in passing through a diversions orks into the canal.

During the settling, samples were taken at four depths. The concentrations f these samples were used to plot the profiles shown as solid lines in Fig. 12. 'or the purpose of comparison, the settling velocity distribution of the origi- al clay particles was obtained by means of a pipette analysis of a suspension f clay and deflocculating agent in distilled water. From the settling distri- ution, the dotted profiles of Fig. 12 were determined. These profile show ow the particles would settle without flocculation.

On the basis of the dotted profiles, one would expect that the clay would be arried in suspension indefinitely. Therefore it would pass through the canal. n fact, however, most of the clay would have settled to the bottom of the canal n one hour.

5. CONCLUSIONS

The principal conclusions based on the research are as follows:

(1) In order to predict the settling of particles in a flowing suspension, it is necessary to know the properties of the flow and the settling properties of the individual suspension. In general, it is best to determine both by direct measurement. If a choice must be made, it will often be better to determine the properties of the flow by calculation or reasonable as- sumption, and to measure the properties of the suspension.

(2) In the initial stages of studying a suspension, it is profitable to obtain a settling velocity distribution by means of a pipette analysis or compara- ble experiment. When flocculation and hindered settling are negligible this distribution can be used in an estimate of removal; when they are not, the results of the analysis are still useful in comparing suspensions.

(3) A multiple-depth pipette analysis can be used to study the effect of hin- drance, flocculation, and turbulence on the settling. For quiescent sett- ling, the analysis should include measurements of the mean settling ve- locity, the standard deviation of settling velocities and the particle concentration at various depths and times. For turbulent settling the analysis should include these three measurements plus any additional measurements related to the turbulence.

(4) Whenever the settling in the flowing suspension can be approximated by settling in a quiescent or turbulent settling tube, the concentration pro- files from a multiple-depth analysis can be used to calculate removal.

(5) When a suspension flows through a settling tank, the removal of parti- cles may depend upon detention time, overflow rate, or both. Whether one or the other is more important depends primarily upon the settling properties of the suspension. For quiescent settling in ideal tanks, a

multiple-depth settling analysis will indicate the relative import overflow rate and detention time.

(6) Settling in a hydraulic model will not be similar to settling in the type unless a suspension with scaled settling properties is used model. When this is not possible, two approaches are available. the model is used to predict the properties of the flow, and separ periments are used to measure the properties of the suspensions These properties are used to calculate what the removal will be predicted prototype flow. In the other approach, the prototype su pension settles in the model. The settling properties of the susp are studied separately and these properties are considered in us model results to predict prototype results.

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SUGGESTED LEGISLATION ON FLOOD PLAIN REGULATION[a]

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ABSTRACT

Adequate statutory authority for city, county and regional planning is lack-
g in many areas. Such authority must be provided if communities are to
gulate the use of flood plains in a manner that will limit and tend to reduce
e damage from floods. This problem concerns all levels of government.

SYNOPSIS

The rapid growth of urban and suburban areas has caused rapid expansion developments into the flood plains of this country. The increase in flood mageable improvements has outstripped all efforts to provide flood control flood protection.

An increasing awareness of the necessity for limiting the flood damage po-ntial is needed at the local level. The preparation of plans for the wise use flood plains, and the adoption of regulations to ensure such use are re-ired for proper and wholesome community development.

Not all states provide adequate statutory authority for city, county and gional planning. Such authority should be provided. In addition, states ould provide controls over construction of dams, levees and encroachments floodways, should collect basic flood data, and should advise local planning encies regarding flood plain regulation problems.

Federal agencies should expand basic data collection programs, observe cal flood plain regulations, and require enactment of appropriate flood plain gulations by local cooperating parties as a prerequisite to qualifying for rtain types of federal programs.

ote: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2315 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 12, December, 1959.

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INTRODUCTION

The rapid growth of urban and suburban areas since World War II ha
an outstanding feature of the economic boom that has taken place during
post war years. This growth has been stimulated as a result of the sub
amount of building during the depression years of the thirties, the war
strictions of the early forties, the very rapid increase in population fol
the war, and the favorable employment and earning levels that have exi
during the past fourteen years.

The land requirements to accommodate this growth have been greatl
larged by the trend toward sprawling one story houses, and by septic ta
sanitary system drainage requirements in suburban locations. These t
lead to lots two to three times larger than those that were considered a
forty or fifty years ago. The greater land requirements have placed a
mium on areas of flat or gently sloping topography which can be develo
most economically. Undeveloped flood plains fulfill these requirements
ticularly in the regions of the country where topography tends to be rol
or hilly.

For these reasons thousands of new homes have been constructed in
subject to flooding. In many instances subdivisions have been develope
full knowledge of the dangers involved, and the developer has either ign
these dangers or considered them so remote that little consideration n
to be given to them. In addition, the planning commissions or other bod
with responsibility for approving new developments frequently have pai
attention to flood danger as an element to be considered in appraising t
suitability of a site for residential, business or industrial development.
other instances the approving bodies have recognized the danger but ha
known how to deal with it, except to bear with it and hope for the best.

Industry is a bit more cautious than many other developers in select
sites for new developments or in expanding old installations and will tr
avoid flood dangers or will protect against them. However, there are n
ous instances throughout the country where such dangers have not been
avoided. Many times industries have exposed themselves to possible d
because of other considerations that were deemed to be more pressing
significant to their operations.

In addition to being exposed to flood danger, many developments also
croach on the floodway, reduce the efficiency and capacity of the floodw
consequently increase flood levels and damage to themselves and other

In some areas, as flood heights have been reduced by flood control r
voirs and the danger of flooding diminished, developments have encroa
farther into the flood plains and again have become subject to damage f
lesser floods. Such use of the flood plains tends to nullify many of the
from flood control projects.

It has been pointed out by White(1) that although the Federal govern
has spent over $4 billion for flood protection since 1936 there has been
decrease in the mean annual flood loss. On the contrary, flood damage
tended to increase significantly. He attributes much of this increase to
great increase in use of the flood plains.

If the flood damage potential is growing and outstripping the relief b
obtained from flood control and protection projects, why are constructi
measures not being taken to more effectively control the improvements
create and increase the damage potential? The answer is in the lack o

recognition of the problem at the local level, lack of adequate legislative authority to deal with the problem, failure to use existing authority, and lack of adequate knowledge for dealing with the technical phases of the problem.

Local Regulations

If the problem of reducing the flood damage potential in our flood plains is considered to be oneof controlling the unwise actions of people, its solution will involve the use of the police powers of the states and the extension of those powers to local governing officials. The use of police powers is still a realm that is reserved for the states and local governments under the terms of the Constitution. It therefore devolves upon local officials, who are in most direct contact with the people involved, to perform the primary policing functions.

There has been a growing awareness of the responsibility of local units of government to plan for the wise use of flood plains. During the past five years an increasing number of communities have included flood plains as special use areas in their community plans. However, the number of such communities is pitifully small compared to the total number of communities engaged in planning.

The lack of adequate statutory authority to permit proper planning is a factor that is retarding progress in some areas. The use of improper techniques or ineffective regulations provides inadequate controls in other areas.

The legislatures of 45 states (including Alaska and Hawaii) and the District of Columbia have expressly authorized community planning. Community-type planning on a regional basis has been authorized by 31 states.

Although planning enabling statues have been enacted in 45 states, in 14 states there is no express statutory authority granted to extend planning activities beyond the corporate limits of cities; in 24 states there is no express statutory authority granted for final approval or adoption of a comprehensive plan by the city council or other local legislative body after it has been prepared.(2)

Some of the five states (Arizona, Florida, Missouri, Texas, and Wyoming) that do not have general laws authorizing the establishment of local planning agencies with power to prepare comprehensive community plans for urban areas, have provisions made for planning by other means. In Florida, authority to establish city and county planning commissions is found in special legislative enactments. Missouri provides for county planning under certain circumstances; and in Texas, city planning commissions are authorized by the charters of individual cities.

In addition to deficiencies in planning legislation, there are also deficiencies in authority for zoning and regulating subdivisions, particularly on a county or regional basis. State legislatures have passed enabling acts giving zoning powers to about half of the 3,000 counties in the United States.(3) In some states all counties may zone; in others, only the more populous counties may do so.

It is readily seen that the lack of adequate authority to plan and zone will restrict the use of flood plain regulations in large sections of the country.

Of the communities that have authority to plan and zone, not all are taking advantage of the powers available to them and some are even hostile to their use. Such hostility is usually justified on the theory that community planning,

zoning and other regulations infringe on the right of the individual to exe
free choice in the use of his property and therefore should be opposed fo
preservation of individual liberty. Fortunately, such attitudes are held
decreasing minority as it becomes more evident that an increasing popu
density requires closer living associations with neighbors and that unre
ed individual liberty frequently produces conflicts between individuals o
dividuals and groups of people to the detriment of the general welfare.

The need for authority to plan land use and the need for courage and
termination in taking steps to guide such use along wise paths indicate t
there are four primary requirements for carrying out regulation of floo
plains at the local level:

1. State enabling legislation to empower communities to prepare com
munity development plans.

2. State enabling legislation to empower communities to enact zoning
other regulations with flood plain provisions.

3. The will to prepare the necessary plans and enact the necessary o
dinances.

4. The will to enforce the ordinances after enactment and to resist g
ing exceptions to them.

These basic principles appear simple enough but their application us
is difficult. In zoning a flood plain to serve the needs of the community
best manner possible, with a minimum of danger from flood damage and
the future growth of the community considered, a careful analysis of the
flooding situation is required. A distinction between the general "flood
and the floodway must be made. The flood plain is the area adjoining th
river or stream which has been or in the future may be covered by flood
water. The floodway includes the channel of a river or stream and thos
tions of the flood plains adjoining the channel which are reasonably requ
to efficiently carry and discharge the flood water or flood flow of the ri
stream. It is readily seen from these definitions that the floodway must
be obstructed if its efficiency is to be preserved. On the other hand, the
ficiency of the floodway may be increased by such measures as enlargin
channel, removing obstructions, and other works. Such works should re
consideration in some cases in planning the use of the flood plain and w
the economic influences demanding the placing of improvements in the f
plain against the need to restrict flood damages and preserve the degre
efficiency of the existing floodway.

Such problems are highly technical and are usually too involved for
local zoning board to solve satisfactorily. In such situations, help shou
solicited from the local engineering department if adequately staffed to
with the problem, from consultants, and from state or federal water res
agencies.

The action required by individual cities and counties in obtaining effe
regulation of flood plains has been very ably summarized by Murphy.(4)
points out that local communities need to consider:

1. Forming a planning commission if one does not already exist and
paring a master plan of land use of the community.

2. Reviewing existing ordinances and codes that contain provisions c
cerning development in flood areas, revising them, and adding to them i
are inadequate to regulate development under major flood conditions.

. Preparing zoning ordinances, subdivision regulations, and building es, if not already in existence, so that certain definite and adequate pro- ons for regulation of flood-plain development are included.

. Cooperating with federal and state agencies in the procurement of ded flood data.

. Instituting a program of acquisition of appropriate portions of the most ardous and frequently flooded lands.

. Instituting a program of annual maintenance of stream channels.

n addition to the actions cited above, the will to do something about the blem and the will to enforce regulations are very necessary ingredients chieving effective results.

State Legislation

n the discussion of local regulation of flood plains it was pointed out that uate statutory authorization for planning, zoning and subdivision control e city, county and regional levels is lacking in a number of states. Such orization by the state is a first step toward enabling local communities ssume the duties that must be carried out at the local level.

n addition to granting authority for local action, the states can take other s to support effective flood plain regulation. This support can be in the n of promotion of local responsibility in flood plain regulation, technical advisory help with flood plain problems, state supervision of certain types evelopments, such as dams and levees, and state control of floodways r certain circumstances.

ack of understanding of flood plain problems, ignorance of the potential d danger in the flood plain, and local pressures to disregard recognized er account for much of the inaction in dealing with flood plain problems. ffective program of contacting planning officials and zoning boards to in- n them of the flood dangers in their communities, of steps they may take ombat those dangers, and of encouraging them to adopt the plans and or- nces necessary can be carried out at the state level. A state agency with or two people, expert in these matters, can serve all the communites in state in promoting better flood plain regulation.

he determination of the floodway requirements and boundaries is a tech- l task that involves a study of past and probably future flood discharges, acteristics of the floodway, and possible modifications of the floodway. ory of past floods is frequently insufficient to furnish a safe guide for lway capacity requirements, and future greater floods must be considered. he technical experience for making such studies will be lacking on most ning boards and on most city engineering staffs. However, most states have water resources agencies with responsibilities for making studies oods and other water problems. These agencies, because of their famil- y with such problems can provide assistance and advice to local officials.

ocal zoning regulations should prohibit encroachments in floodways and e regulations should be supported by state encroachment laws. In addition, tate encroachment laws should also protect those floodways not protected cal laws. Local planning and zoning officials may find it advantageous quire the approval of state agencies as a prior condition to granting ex- ons to floodway zoning ordinances.

State laws should prohibit encroachment on floodways and should for adequate technical review and approval of all structures, such a dams, pipelines, etc., that must be constructed in floodways. All fil cavating, or other constructions that might adversely affect the effic capacity of the floodway should be prohibited by legislation and pena should be provided for violations.

Dams, which can have a high hazard factor and can be a serious developments on flood plains and to public safety, lack adequate reg many states. Only 45 states require permits for the construction of require approval of plans before construction, and only 35 provide f tion after construction. Even though inspections are made, many st adequate statutory authority to order the abatement or removal of h conditions or to assess penalties for noncompliance with orders or measures.

Levee systems protecting urban areas frequently suffer from ina design and maintenance with consequent danger to life and property. such structures often lack a responsible owner to provide necessary ance. This situation can best be remedied through state inspection requirement that local communities provide adequate maintenance. local maintenance is deficient, the maintenance should be performed state and charged back to the local community.

Flood data can best be gathered and analyzed at the state level b full time specialized technical staffs, which cannot be financed by lo of government, can be provided for such purposes at this level. In the restrictions of local governmental boundaries are not limiting fa the extent and scope of investigations. The controlling features tha flood heights are frequently at a considerable distance beyond the u county limits that limit local jurisdictions.

It would appear that legislation at the state level should provide:

1. Statutory authority for the creation of local planning agencies to prepare comprehensive community plans including designation of plain and floodway areas.
2. Statutory authority for enacting zoning and subdivision contro ces.
3. Enactment of floodway encroachment laws and regulation of c in floodways.
4. Enactment of laws governing construction and maintenance of levees.
5. Establishment of a state flood control or water resources ag regulatory powers, authority to make investigations, collect basic d pare flood-evaluation reports, provide advisory service to local pla agencies, and to cooperate with federal agencies.

The role of the state should be to support the local governmenta any policing actions necessary for local regulation, and to provide lation in those matters that require uniformity of action for justice tical operation and in those situations that transcend the limits of l ernmental jurisdictions.

Federal Legislation

e historic approach of the federal government to the problem of reducing damages has been through the construction of flood control or flood pro-n structures. Little effort has been exerted to prevent the creation of ood damage areas or to encourage local responsibility in this field. ver, there is an awakening at the federal level to the need for preventing eation of new sources of flood damage in addition to continuing the tra-al effort to provide protection to old areas.

e Tennessee Valley Authority in its recent report "A Program for Re-g the National Flood Damage Potential" cites the futility of continuing to le structural measures for flood reduction or flood protection in certain and at the same time permitting new damageable developments to take at an even faster rate in other areas.

e TVA report suggests a three-pointed plan for placing more emphasis control of the growth of the flood damage potential:

1. An expanded program for collection, analysis and wider use of flood a.
2. A broad approach to flood damage abatement which includes preven-ive as well as corrective measures.
3. More active participation by state and local governments in solving flood problem.

implement the program the TVA recommends that:

1. A national flood damage prevention policy should be developed. The ectives of the policy should be (a) to prevent the unnecessary spread of ldings and other improvements to new flood hazard areas, combined with tinued efforts to alleviate damages in those flood hazard areas which e already been developed, and (b) to encourage appropriate state and al bodies to assume the responsibility for initiating and developing plans l programs for local flood damage abatement through construction of tective works and establishment of land use controls as necessary. The icy should also provide a basis for technical and financial assistance by eral agencies.
2. If protective works are economically justified as a part of the local gram for flood damage abatement, the federal financial contribution to h projects should be contingent upon the requirement that the city or te initiate and prepare engineering plans of the proposed works and that plans be submitted to an appropriate federal agency for review and roval.
3. As a prerequisite for the contribution of federal funds to local flood tective works, state and local governments should be required to adopt l administer flood plain zoning and other such controls as are needed to vent the unnecessary spread of buildings and other improvements in eas subject to flood damage.
4. Provision should be made to insure that all federal agencies having ponsibility related to site selection or financing of physical structures erve local flood plain zoning and other standards in carrying out their ponsibilities, and in cities without land use controls but subject to flood nage require establishment of such controls before federal programs approved and put into execution.

5. An expanded and systematic program of collecting and analyz basic flood information by existing federal agencies should be autho which will meet the needs of states, local communities, industries others in adjusting to flood conditions.

6. For the purpose of providing cities and communities with bas information on which to prepare plans for growth and development the prevention of local flood damages, appropriate federal agencies be authorized, where such authorization is necessary, to prepare u quest local flood studies which would describe and analyze the loca situation.

The TVA recommendation, in addition to providing for greater fede tivity in collecting basic data and providing basic flood information to communities, would make flood plain regulation a requirement for qua for certain types of federal assistance. This requirement would provi siderable stimulus to local regulation.

The Chief of Engineers, U. S. Corps of Engineers, has recently inst all the Division and District offices to give greater consideration to fl plain regulation. He directed that survey reports of the Corps of Engi will discuss adequately the problems of encroachments on channels an velopments in flood plains. Where encroachments and developments c expected, appropriate practicable provisions are to be included in pro plans of improvements and in requirements of local cooperation to pr flood-carrying capacity of channels and floodways.

These moves by federal agencies are in the right direction and in g need no legislation to place them in effect. However, there are defici at the general policy level that detract from their effectiveness. A po laration by Congress that would endorse the recommendations of the T the directive of the Chief of Engineers and extend the recommended p to all federal agencies would greatly assist in promoting better flood regulation.

SUMMARY

Perhaps the greatest hindrance to accomplishing more widespread plain regulation is the lack of adequate authority for planning and zoni city, county and regional levels in many states. The acceptance of pla and zoning methods for flood plain regulation is still in its infancy and greater educational effort is needed to bring about greater use of thes for reducing the flood damage potential.

State laws to provide adequate planning and zoning authority should acted promptly where necessary. Such laws should be made adequate city, county and regional levels.

Although all states have provisions for administering water resour tivities in some state agency to some degree, many of the agencies la quate powers to deal with flood problems, to collect basic data or to a regulations. In addition, many are inadequately financed and staffed. states' effort should be to suuport local flood plain regulations, collec flood data and prepare flood evaluation reports, provide advice and te service to local planning agencies in flood plain matters, administer s floodway-encroachment and dam-safety provisions, provide levee insp and require adequate levee maintenance for urban areas, and coopera federal agencies.

The federal government should adopt a policy that will require local regula-ion of flood plains where indicated in federal flood control projects, enlarge he program for the collection of basic flood data, prepare flood-evaluation eports particularly in regions that cross state boundaries, and establish cri-eria for determining the extent to which loans on private construction in flood lain areas would be guaranteed by federal agencies.

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IMPROVED TUNNEL SPILLWAY FLIP BUCKETS

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SYNOPSIS

This paper discusses several buckets, of the type used to deflect or flip [tu]nnel spillway discharges downstream, in terms of their desirable or unde[s]irable features. Several new types of buckets developed from hydraulic [m]odel tests are then described. Using data from these tests, dimensionless [cu]rves are presented to aid in determining the jet trajectory length, the [s]preading of the jet, the tail water drawdown at the bucket, and the pressures [o]n the floor and side walls of the bucket.

INTRODUCTION

A tunnel spillway is composed of two basic parts—an upstream spillway [cr]est, free or controlled, and a downstream tunnel, part of which is sloping [an]d part near horizontal. From the standpoint of economy the tunnel diameter [m]ust be kept to a minimum. Since the tunnel is never allowed to flow full be[ca]use of the possibility of siphonic action producing dangerous flow conditions, [it] is necessary to keep flow velocities high and to prevent turbulent areas in [th]e tunnel. Spillway tunnels are usually designed to flow from 3/4 to 7/8 full [at] maximum discharge, making the outflow at the tunnel portal relatively deep. [T]he combination of depth and velocity produces the highest possible concen[tr]ation of energy and increases the difficulty of obtaining satisfactory flow [co]nditions where the flow spills into the river. As an example, on the Glen [C]anyon tunnel spillways,[1] the maximum discharge of 276,000 cfs produces [5]9,000 horsepower per foot of width at the tunnel portals. On Grand Coulee,[2]

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1. See Table 1, for references 1-8.

an overfall spillway, where the maximum discharge is 1,000,000 cfs, gy per foot of width is only 15,650 horsepower, or one-tenth that on G Canyon.

If it were feasible to construct an efficient hydraulic jump stilling the end of one of the Glen Canyon tunnels, the basin depth, from apro water elevation, would need to be 170 feet deep. The hydraulic jump would be over 1,000 feet and would require a basin 700 to 800 feet lo more. Basin appurtenances such as baffle piers could not be used ef because the high entrance velocity, 165 feet per second, would produ tation problems. The cost of a structure this size would be prohibiti is readily seen why other types of structures are used at the end of t spillways. Buckets have been the most common of these structures probably derived from the slight upturns placed at the base of early spillways. It is not clear whether the designers intended that these b operate free or submerged. In some cases, the upturn was too sligh duce a measurable effect on a thick jet, but probably the intended pu to deflect the jet downstream to prevent undermining of the spillway Buckets of this type are referred to variously as "ski-jump," deflect fuser, trajectory, or flip buckets. For ease of expression, the term bucket will be used in this paper.

Flip buckets are not a substitute for energy dissipators because s bucket is inherently incapable of dissipating energy within itself. Th of a flip bucket is to throw the water downstream where the river-be which is usually certain to occur, does not endanger the safety of the powerplant, or other structures including the flip bucket itself. In ac ing this primary function, buckets are also designed to spread the flo as much of the downstream channel as is considered desirable in ord duce riverbed damage as much as possible. The jet trajectory is mo necessary to cause the jet to impinge on the tail water surface at the location, and when possible, the steepness of the jet trajectory at the impingement is selected to produce horizontal and vertical velocity c nents which produce most favorable flow conditions in the river chan

Although with the present state of knowledge it is impractical to g the design of flip buckets, it is intended in this paper to present certa facts which have been found to be true as a result of extensive hydra

Table 1

DESCRIPTION OF PROJECTS

Reference No.	Name and location	Agency	Maximum discharge	Fall max headwater to bucket invert	Tu dime
1	Glen Canyon Dam Colorado River Storage Project, Arizona	Bureau of Reclamation	276,000 cfs	588'	2 tunnel each 4
2	Grand Coulee Dam Columbia Basin Project Washington	Bureau of Reclamation	1,000,000 cfs	420'	Overfall 1,650'
3	Hungry Horse Dam Hungry Horse Dam Project Montana	Bureau of Reclamation	50,000 cfs	488'	31' diam
4	Yellowtail Dam Missouri River Basin Project Montana	Bureau of Reclamation	173,000 cfs	512'	20.5' di horses
5	Yanhee Dam, Thailand	Kingdom of Thailand Ministry of Agriculture Royal Irrigation Department	212,000 cfs	402'	2 horses 37.08'
6	Wu-Sheh Dam Taiwan, China	Taiwan Power Company	66,000 cfs	387'	27' diam
7	Fontana Dam North Carolina	Tennessee Valley Authority	180,000 cfs	423'	2 tunnel each 3
8	Trinity Dam Central Valley Project California	Bureau of Reclamation	24,000 cfs	475'	20' diam

sting and prototype observation. It is hoped that extension of the ideas presented, in the form of discussions, will help to provide a better understanding the requirements necessary for improved flip bucket design.

Bucket Design Problems

It is usually difficult or impossible to predict the flow pattern to be expect- from a particular bucket by mere inspection of the bucket shape. Because variations in velocity and depth, the spreading and trajectory character- tics of a given bucket can be determined only by testing in a hydraulic model. nce the authors have had the opportunity to test various types of buckets and observe or hear first hand of their performance in the field, the findings of ese tests should, therefore, be of interest to designers who must often se- ct a bucket type before the hydraulic model tests are made.

In the course of developing and improving bucket designs, a number of fficulties have been found and overcome. The following examples indicate e problems which may be encountered in bucket design and which may not be nerally known.

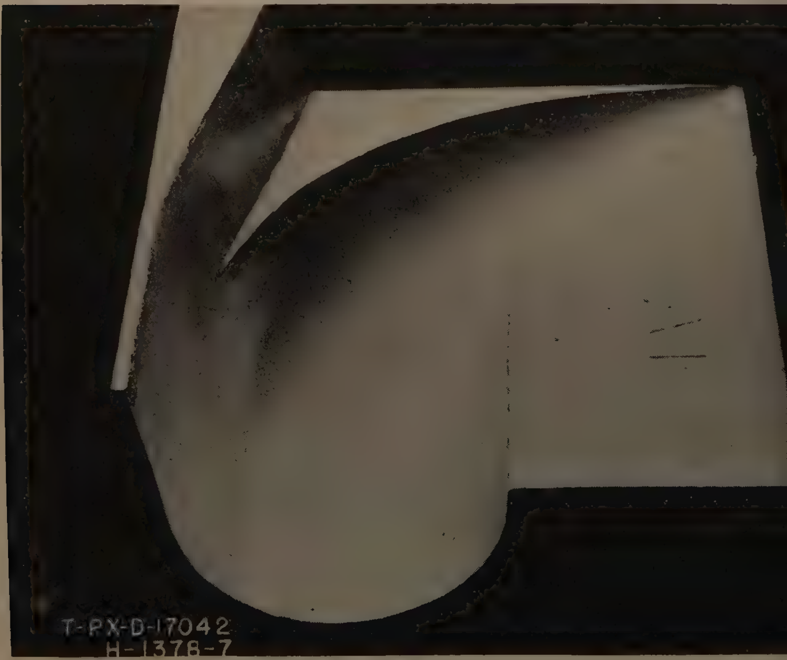
The flip buckets on the tunnel spillways at Hungry Horse Dam[3] and Yellow- il Dam[4] of the Bureau of Reclamation projects, and Yanhee Dam[5] and Wu- ieh Dam[6] being built in Thailand and Formosa, respectively, are similar and e what may be called a "standard" type. The buckets are placed down- ream from a transition which changes the circular or horseshoe-shaped nnel to a flat bottom to correspond to the flat bottom of the bucket. High ve- city flow in the tunnel makes it difficult to design a short transition; long ansitions are usually costly. If the transition is not carefully designed, and eferably checked by model studies, there is the possibility of dangerous sub- mospheric pressures occurring in the corners. The transition, therefore, comes as much of a design problem as the bucket.

The Fontana Dam[7] spillway buckets do not have an upstream transition. ne bucket inverts are circular, the same as the tunnel inverts, Fig. 1.* The ckets were shaped by trial in a 1:100-scale model tested in the TVA Hy- aulic Laboratory. The curved surfaces of the finally developed buckets uld not be defined by ordinary dimensioning or even by mathematical uations. That the buckets were well designed has been proved by subsequent eration of the structure, but the methods necessary to convert the model di- ensions at a scale of 1:100 to prototype dimensions were quite laborious. ecause of the high velocity flow in the bucket, dimensions taken from the odel could not be "scaled up" directly. Any small irregularity or misaline- ent when multiplied by 100 could have been sufficiently large to produce vitation in the prototype bucket. It was, therefore, necessary to convert the mensions to a 1:10-scale bucket, and after smoothing these, to convert the rrected dimensions to a 1:1 scale.

On some buckets, particularly those on foreign dams, a serrated or toothed lge has been placed at the downstream end of the bucket. The teeth are to ovide greater dispersion of the jet before it strikes the tail water surface. gh velocity flow passing over the sharp edges may produce cavitation mage on the concrete surfaces.

Photographs from Technical Monograph No. 68, Hydraulic model studies, Fontana Project, Tennessee Valley Authority.

Bucket used at Tunnel 1 outlet



Bucket used at Tunnel 2 outlet

Fig. 1. Fontana Dam Spillway Flip Buckets

The problem of draining a tunnel which has a flip bucket at the downstream provides a challenge in design. The drain must be placed in a surface exed to high velocity flow. Even though it is possible to design or develop in laboratory a drain opening which will not produce cavitation pressures, it difficult to obtain field construction to the necessary tolerances to prevent itation from occurring. An ideal bucket design would be self-draining and ld not present a cavitation problem at the drain structure.

Improved Bucket Designs

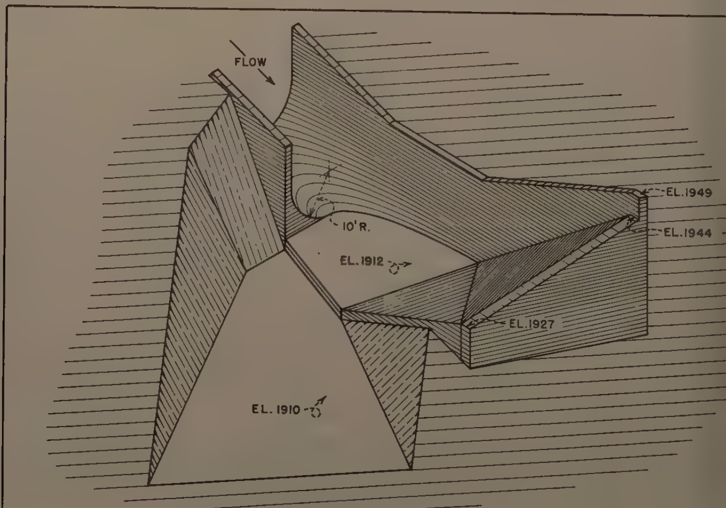
Recently a number of tunnel spillway flip buckets have been developed in Bureau of Reclamation Hydraulic Laboratory that seem to offer simple but y effective methods of directing the flow away from the structure and which o overcome, in part, the difficulties described in the preceding paragraphs. hough no single bucket eliminates all of the undesirable features of a ket, the use of the principles described in the following pages will help the igner to provide an improved bucket on a particular structure. To review, ideal bucket should provide (1) easy drainage of the tunnel, (2) a bucket pe which can be defined and expressed in prototype size by ordinary dinsioning on ordinary drawings, (3) no need for an upstream transition, and an impingement area which may be shaped, by simple additions to a basic ket, to fit the existing topographic conditions. Some of the buckets deibed are unique and probably cannot be generally used without some adapton; however, the others are basic in type and have only minor additions to omplish some specific function.

One of the unique designs was the Trinity Dam[8] spillway bucket developed a 1:80-scale model. The spillway tunnel enters one side of a wide shallow er channel and the flow tends to cross the river diagonally. It was necesy to discharge the flow into this channel without creating excessive eddies t might erode the riverbanks or cause disturbances in the vicinity of the verhouse tailrace. The spillway is an uncontrolled morning-glory and, sequently, the flow can vary from a few second-feet to a maximum of 000 cfs. The velocity at the bucket is 122 feet per second. Because small ws may occur for days, it was desirable for low flows to leave the bucket close to the riverbed elevation as possible to prevent excessive erosion r the base of the structure. On the other hand, large flows should be ped downstream away from the structure with as much dispersion as possi- to prevent erosion and induced eddies from damaging the structure. In the al flip bucket, a hydraulic jump forms in the bucket for small flows and the ter dribbles over the bucket end and falls onto the riverbed. This could se erosion which would undermine the structure. When the jump is first ept out of the bucket, the jet usually lands near the structure and erosion undermining of the structure may still occur. At Trinity Dam, the founion conditions at the end of the tunnel were such that it was deemed necesy to protect against the possibility of erosion and undermining. In order to ce the bucket near riverbed level, the semicircular channel constructed nstream from the tunnel portal was curved downward in a trajectory curve, the flip bucket structure was placed at the end, Fig. 2. The flip bucket face consisted of three plane surfaces so placed that they spread and shape jet to fit the surrounding topography. Large flows are spread into a thin et having a contact line with the tail water surface a considerable distance

downstream, Fig. 3A. However, even small flows are thrown downst well away from the base of the bucket.

A training wall was used to prevent spreading of the jet on the higl side, of the bucket. There was no wall on the low or river side of the At flows less than 1,000 cfs, a hydraulic jump formed over the horizo surface and part way up the slope of the bucket; the flow spilled out o side of the bucket into the river channel. The open side of the bucket 4 or 5 feet above the river. Had the flow been confined on both sides forced to spill out the end, the drop would have been over 40 feet and al protection of the bucket foundation would have been required. At d greater than 1,000 cfs, the jump swept out of the bucket without hesit with sufficient velocity that the flow was carried well downstream aw the structure. As the discharge increased, the jet was flipped farthe stream and became increasingly dispersed. The long contact line be jet and the tail water reduced the unit forces on the tail water, and th induced at the ends of the contact line were thereby found to be a min Since one side of the bucket is entirely open, the bucket is self-drain Other advantages of this design are that the bucket may be defined fo type construction with a few simple dimensions, and no curved or wa forms are necessary for prototype construction.

Another unusual type of flip bucket was developed for the Wu-Sheh tunnel spillway. Construction schedules and geologic conditions in th made it necessary to modify this bucket from the standard type descr earlier in this paper. After the line of the tunnel had been establishe construction of the tunnel started, it was found necessary as a result tests to change the direction of the flow entering the river channel. rock slides, during the diversion period, made it necessary to constr taining walls in the tunnel portal area which restricted the length of bucket. Hydraulic model studies were made to determine how much the jet was required and whether the turning could be accomplished i tunnel. The tests showed that it was undesirable to turn the tunnel ar

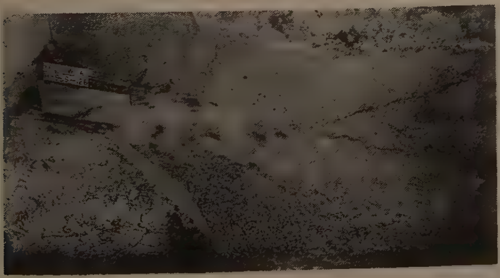


DISPERSION FLIP BUCKET

Fig. 2

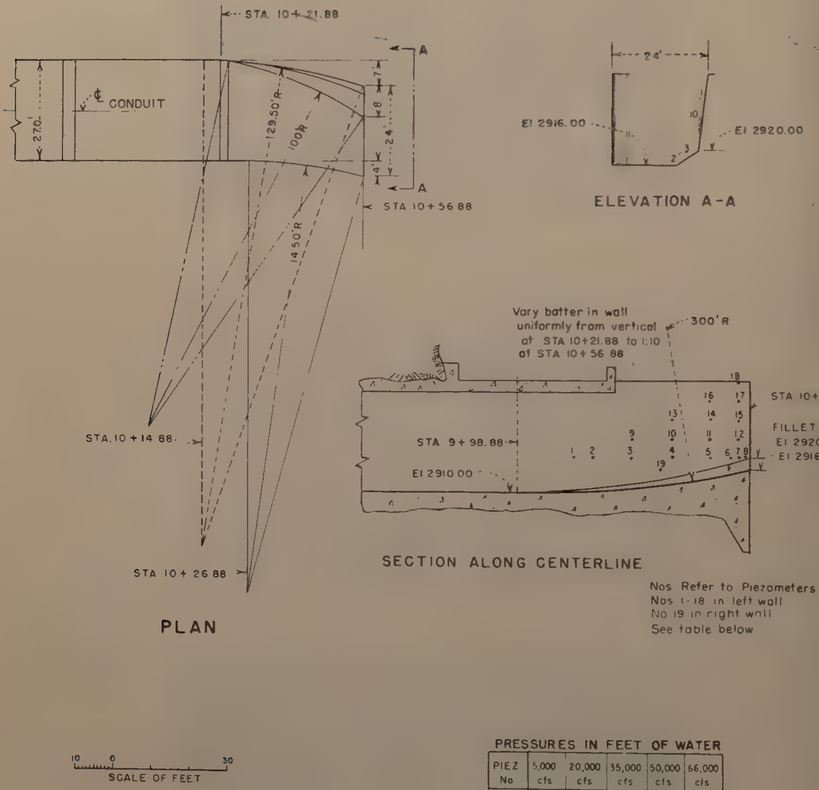
Fig. 3A. Dispersion-Type Flip Bucket

Hydraulic Jump in Basin—Discharge 12,000 cfs



Q = 13,000 cfs, Basin Acts as Flip Bucket

Fig. 3B. Combination Hydraulic Jump Basin-Flip Bucket



WU SHEH DAM
TUNNEL SPILLWAY
HYDRAULIC MODEL STUDIES
1 41 25 SCALE MODEL
RECOMMENDED BUCKET

PRESSURES IN FEET OF WATER

PIEZ No	5,000 cfs	20,000 cfs	35,000 cfs	50,000 cfs	66,000 cfs
1	–	–	+9 0	+28 4	+50 2
2	–	+0 2	20 0	43.6	66 6
3	0	12.1	43.2	70 3	90 9
4	+2.3	23 3	50 6	70 7	89 0
5	6 4	36.5	59 2	72 7	84 0
6	6 1	30 8	45 4	53 6	62 0
7	4.4	23 8	35 0	41 2	46 8
8	2 1	13.9	19.4	22 6	26.3
9	-0.7	0	11 7	38 8	65 6
10	0	6 4	26 8	51 2	74 1
11	+0.9	20.5	42.4	58 0	70 2
12	2 7	17.7	32 6	41 0	46 9
13	–	1 0	4 3	21 1	47 5
14	–	-2.1	+5.2	21 1	37 1
15	–	+8 4	17.6	28 2	36.2
16	–	–	0.3	8 0	24 7
17	–	2 6	8 6	16 1	26.8
18	–	–	2.5	4 0	9 0
19	+3 6	+13 8	19.5	22 5	26 8

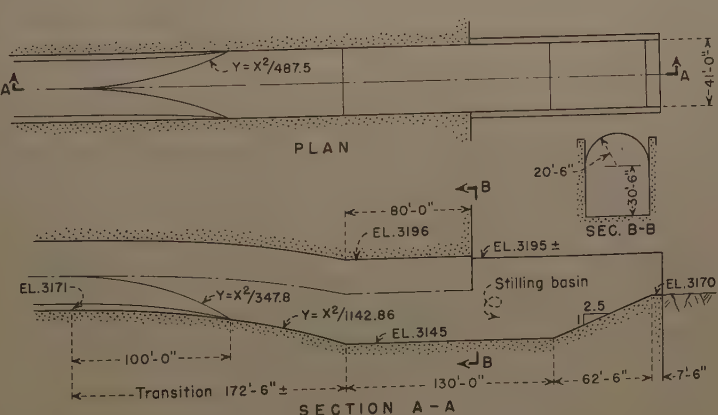
568

Fig. 4

turning should be accomplished in the bucket. The final bucket determined from model studies used curved walls to turn the flow—a batter in the left wall to prevent congestion in the bucket and reduce hydraulic loads at the larger discharges, and a fillet at the junction of the left wall and floor to smooth up or control the jet undernappe, Fig. 4. The resulting bucket was "tailor-made" to direct the flow to impinge near the middle of the river channel and to obtain the greatest dispersion possible at all discharges. The surfaces in this bucket could also be defined by ordinary dimensioning.

Piezometers placed in the side walls of the bucket showed above-atmospheric pressures at all discharges. The maximum pressure recorded on the left wall was 91 feet of water, Fig. 4. Before the wall was battered, the maximum pressure probably would have been much larger due to a more direct impact on the converging wall.

The Yellowtail Dam tunnel spillway flip bucket is a dual purpose bucket similar in some respects to the standard buckets. The tunnel is a curved bottom horseshoe-type conduit. Two hundred fifty feet upstream from the portal, the tunnel changes to a flat bottom horseshoe conduit, and the invert drops 26 feet by means of a combination transition-trajectory curve 172-1/2 feet long. The bucket has a flat horizontal floor 130 feet long and a 62-1/2-foot-long upward sloping sill, Fig. 5. At spillway flows up to 12,000 cfs, a hydraulic jump forms in the bucket and relatively quiet water is discharged into the downstream channel. As the spillway discharge increases, the jump moves downstream and at 13,000 cfs sweeps out of the basin, Fig. 3B; for greater discharges and up to the maximum, 173,000 cfs, the basin acts as a flip bucket. The basin or bucket is placed low in solid rock so that discharges in the unstable zone, 12,000 to 13,000 cfs, cannot undermine the structure. This basin was developed in the Hydraulic Laboratory to serve the specific purpose of acting as a hydraulic jump basin for the most prevalent spillway discharges—discharges expected to be exceeded only every 10 years, and acting as a flip bucket to prevent damage to the structures during large floods.



YELLOWTAIL DAM STILLING BASIN

Fig. 5

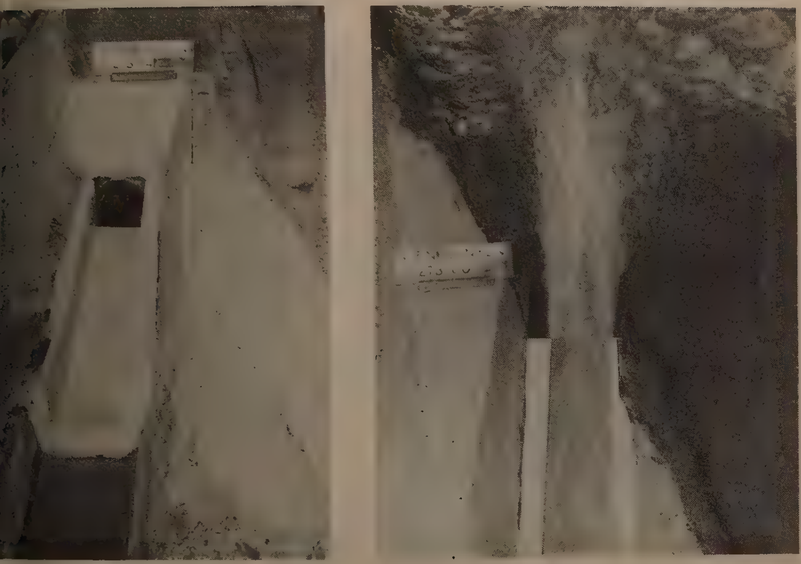
The reason for using the hydraulic jump for part of the discharge range
to protect the river channel against clogging with talus which was prese
the canyon in large quantities and was expected to move if a high veloci
stream contacted it. Following large discharges, it was expected that r
opening of the channel to achieve full power head would be necessary.

The flip buckets for the Glen Canyon Dam tunnel spillways are an ex
of buckets developed to eliminate the tunnel transition and the need for
bucket invert. The buckets at the portals of the 41-foot-diameter tunne
on opposite sides of the river and are aimed to discharge at acute angle
the center of the river. The left bucket is farther downstream than the
Each bucket is designed to handle the maximum discharge of 138,000 cf
velocity of about 165 feet per second. This represents over 13,000,000
power in energy released into the river during maximum discharge.

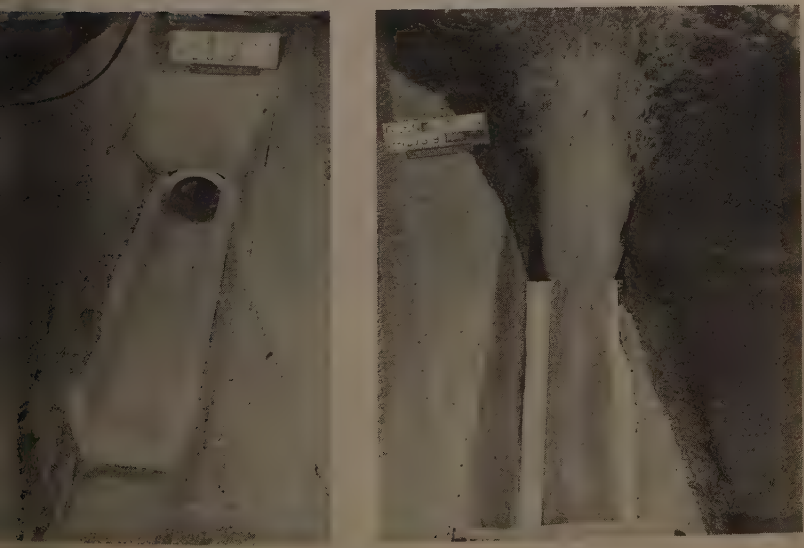
In the preliminary design, there was a 70-foot-long transition betwe
circular tunnel and the rectangular channel containing the flip bucket.
draulic model studies indicated that the transition was too short, and th
atmospheric pressures would be sufficiently low to produce cavitation a
damage to the structure. Two alternatives were developed during the m
studies; one was to use a 100-foot-long transition in which the change i
section was accomplished without dangerous pressures occurring and t
other was to eliminate the transition by continuing the circular tunnel i
downstream to intersect the upward curve of the flip bucket. The latte
scheme was developed and will be used in the prototype structure; ident
buckets will be constructed on the twin spillways. In effect, the transit
the bucket are combined into the bucket structure without complicating
sign of the bucket.

Because the flat-bottom portion of these buckets diverges in plan, s
flows are spread laterally more than for the flat-bottomed bucket. As
charge increases, the rate of spreading decreases so that it is easier t
commodate the jet for flood flows in a relatively narrow channel. Fig.
shows a comparison of the flow from the two types of buckets. In the f
bottomed bucket which is preceded by a transition, the flip curve exten
across the full width of the bucket for its entire length; all of the flow e
at a given elevation are turned simultaneously. In the alternate bucket,
flip curve turns the lower flow elements in the center of the stream fir
gradually widens its zone of influence as the flow moves downstream,
ing in greater dispersion of the jet. In effect, the flow along the center
the bucket is turned upward while the flow elements on either side of th
center are turned upward and laterally. Training walls may be used to
the lateral spreading. In subsequent testing, deflectors were added to
bucket training walls to make the jets conform to the shape of the rive
channel and surrounding topography, Fig. 7.

The flip bucket used on the Flaming Gorge Dam tunnel spillway was
same type as used on the Glen Canyon spillways. The maximum design
for Flaming Gorge spillway is 28,800 cfs; the velocity of the flow at the
of the 18-foot-diameter tunnel is about 140 feet per second. The energ
jet at the flip bucket is equivalent to 1,000,000 horsepower. In operatio
flow appearance of the Flaming Gorge bucket was entirely different tha
of the Glen Canyon buckets. The Flaming Gorge jet was well disperse
lower discharges and became more compact as the discharge increase
8. The Glen Canyon jets were well dispersed for all flows, and the cha
lateral spreading with discharge was not so apparent. In the Flaming

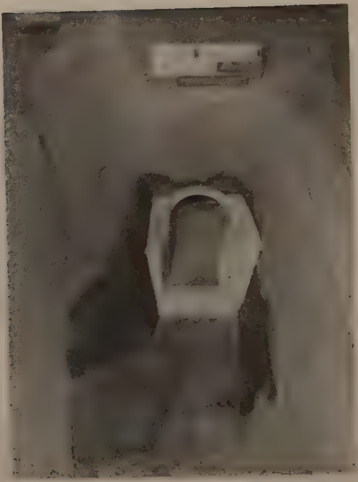


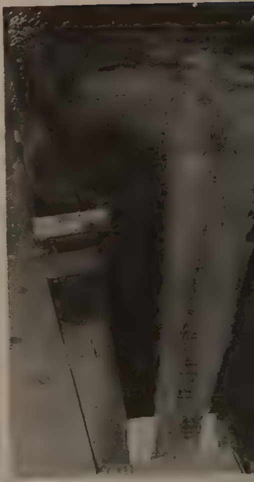
Standard Flat Bottom Flip Bucket
Flow at Froude Number 7.89



Transition Flip Bucket
Flow at Froude Number 5.64

Fig. 6. Glen Canyon Dam
Flip Bucket Studies

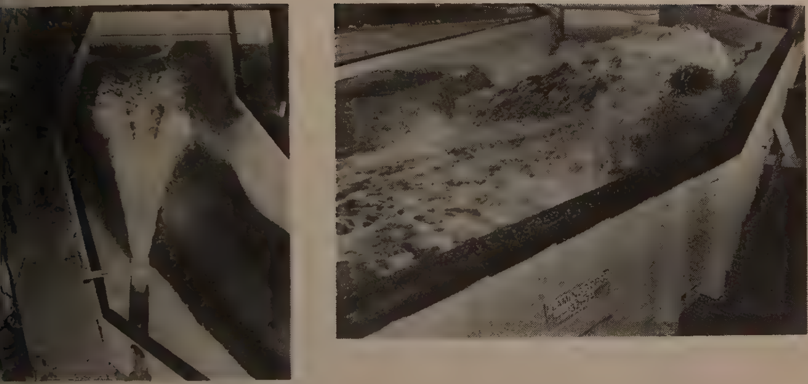




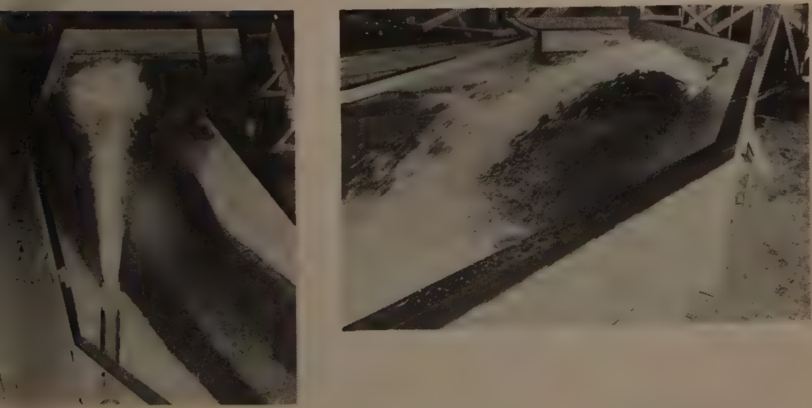
Transition Flip Bucket with Side Wall Deflector
Flow at Froude Number 5.64

Typical Jet Profile
35° Transition Flip Bucket

Fig. 7. Glen Canyon Dam
Flip Bucket Studies



Jet is Well Dispersed at Low Flows.
Froude Number 10.3



A More Compact Jet Forms for Larger Flows.
Froude Number 6.8—Maximum Discharge

Fig. 8. Flaming Gorge Dam

Flip Bucket Studies

35° Transition Bucket

bucket, the water rose on the sides of the bucket at low flows, formin
fect a "U"-shaped sheet of water in which the bottom and sides were
thickness. The vertical sides of the "U" followed the line of the buck
walls after leaving the bucket, while the bottom sheet of water had a
to diverge to either side. The vertical fins had a shorter trajectory
lower sheet and on falling would penetrate the lower jet, tending to sp
disperse it; this can be seen in the photographs in Fig. 8. As the dis
increased, the size of the fins relative to the thickness of the lower s
came insignificant and no longer had this spreading effect. The diffe
the Glen Canyon and Flaming Gorge jets might be explained by the fa
the flow depth for maximum discharge was about 61 per cent of the d
of the Glen Canyon tunnel and 81 per cent of the diameter of the Flan
Gorge tunnel. For a flow 0.61D in Flaming Gorge, the jet was still w
persed.

Both the Flaming Gorge and Glen Canyon buckets were modified b
ing the height of the river side wall. The Flaming Gorge bucket is lo
well above the maximum tail water elevation so that the wall could b
to the spring line of the tunnel invert curve without tail water interfe
The effect was to eliminate the fin that formerly rose along the wall.
spread out evenly to the right and was better dispersed than before.
Canyon buckets are located more closely to the maximum tail water
and to prevent the tail water from interfering with the jet, the river
be cut down to only 5 feet above the spring line of the tunnel invert.
wall remained to train the jet and very little difference in the flow pa
could be detected.

Design Considerations

Tunnel spillways usually make use of part of the river diversion t
The downstream portion of the diversion tunnel becomes the horizon
of the spillway tunnel—the bucket is added after diversion needs have
satisfied. Since the diversion tunnel is one of the first items of cons
it is often necessary, because of time limitations and construction sc
to determine line and grade for the diversion tunnels before the deta
spillway are known. Care in selecting the exact position and elevatio
diversion tunnel, while keeping in mind its ultimate use as a spillway
will help to provide a dual purpose tunnel which will satisfy the temp
well as the final demands with the least amount of modification when
bucket is added.

The following sections cover the items which should be considere
design and which will help to provide a simple bucket structure havi
ble performance characteristics.

Elevation of Bucket Invert

It is desirable to construct the bucket and tunnel inverts at the sa
vation. Since diversion requirements make it necessary to keep the
tunnel low to provide the diversion capacity, the greatest danger is t
tunnel will be set too low for ideal spillway operation. This will req
ing up the bucket lip to prevent the tail water from submerging the b
a general rule, maximum tail water should be no higher than the ele
the center line of the tunnel. If the bucket is set lower, difficulty ma

rienced in obtaining free flow at low spillway discharges. The shape of ail water curve will determine the exact requirements. The drawdown in water elevation at the bucket caused by the ejector action of the jet may affect the vertical placement. Drawdown is discussed later.

v Direction

The bucket center line should be a continuation of the tunnel center line, the portion of the diversion tunnel used for the spillway tunnel should be ight. It is, therefore, desirable to aim the diversion tunnel so that it may sed without change for the spillway tunnel. The tunnel direction should be so that spillway flows will be aimed downriver and so that the design disrge impinges on the tail water in the center of the discharge channel. The should be directed to minimize the diameter of induced eddies at the sides ne jet since these can be very damaging to channel banks. In an ideal argement, the jet will be as wide as the channel so that there will be little irn flow from the downstream tail water.

Fig. 9A shows the angle of divergence of one side of the jet leaving the ket for two types of buckets—the flat-bottom type and the transition bucket d on Glen Canyon and Flaming Gorge spillways. In both cases, the angle ivergence is plotted versus the angle of inclination θ for a range of Froude nbers (of the flow entering the bucket). The flat bottom bucket produced y little change in angle of divergence for a range of Froude numbers or ination angles. The transition bucket showed considerable change in digence angle—from 4^0 to 12^0 for a Froude number range of 6 to 11. Since higher Froude numbers occur at low discharges, the transition-bucket-jet ergence is greatest at low flows. As the discharge increases, the Froude nber becomes smaller and the divergence angle decreases. In most dens this is a favorable characteristic and results in improved river flow ditions for all discharges.

wdown

For the conditions just described, the jet will act as an ejector to lower the water upstream from the jet impingement area. From the Hungry Horse n model tests, 26 feet of drawdown was predicted for 35,000 cfs discharge, it was recommended that a weir be constructed in the powerplant tailrace prevent unwatering of the turbines. Prototype tests made for 30,000 cfs wed 25 feet of drawdown and demonstrated that the weir was indeed necesy. At Hungry Horse the flow leaves the bucket at a 15^0 angle, making the jectory relatively flat, Fig. 10; the jet is as wide as the downstream nnel. The drawdown is maximum under these conditions. At Glen Canyon spillway jets do not occupy the entire width of channel, but the jet trajectois steeper, and the discharge is considerably greater. Hydraulic model ts have indicated that up to 25 feet of drawdown may be expected.

Other hydraulic model bucket tests have shown the drawdown to be apprecie, particularly when the jet occupies a large proportion of the channel lth. No means have been found to calculate the amount of drawdown to be ected, except by making careful measurements on a hydraulic model. Hower, by using measurements obtained on several model studies and from ited prototype observations, the curve in Fig. 9B was derived. It is prented here as a means of estimating the drawdown that can be expected with nnel spillway and flip bucket.

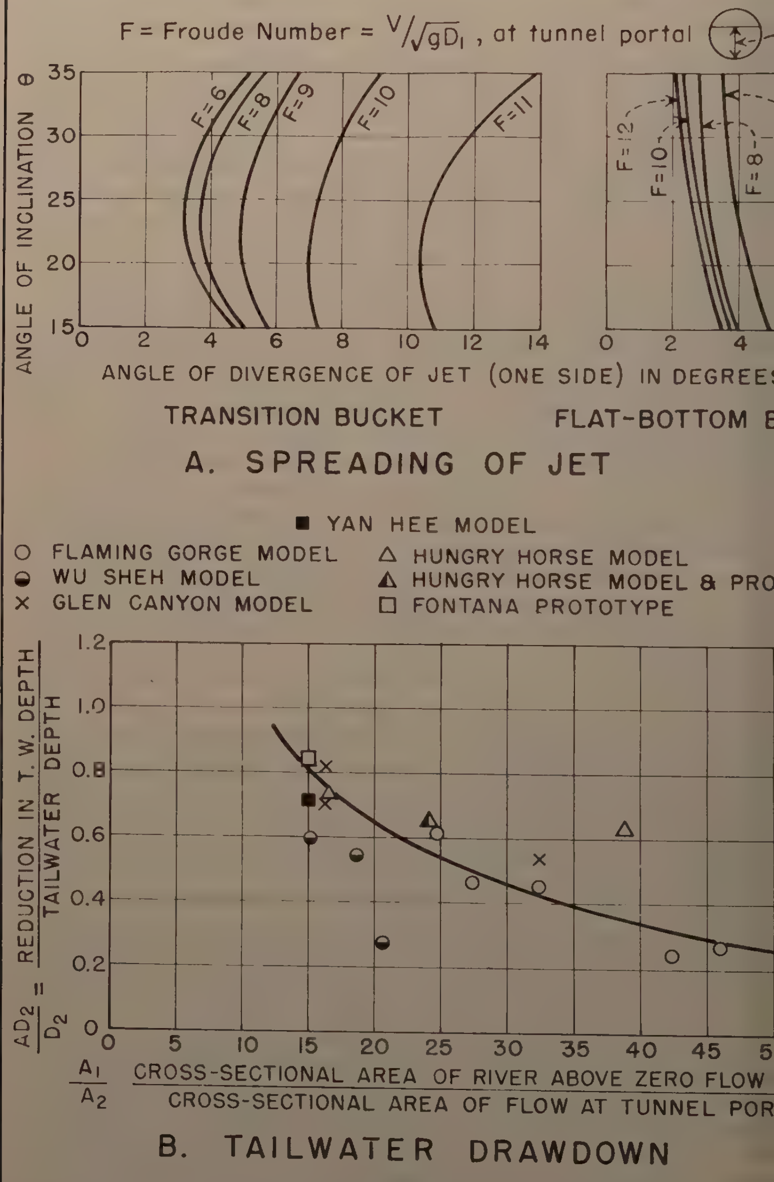
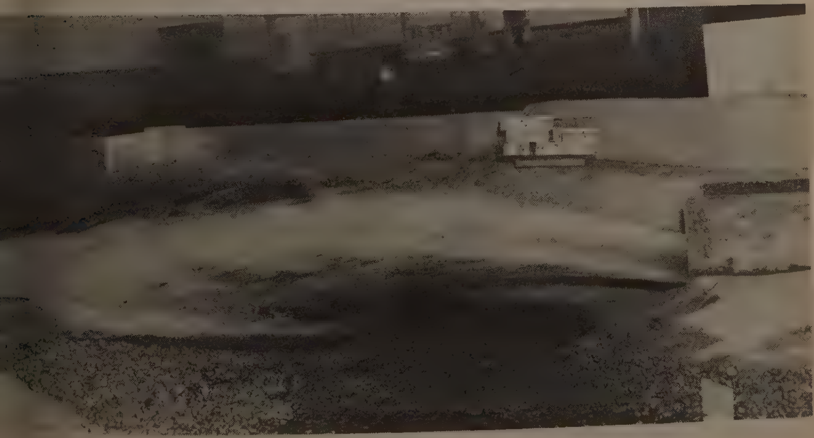

F = Froude Number = V/√gD1 , at tunnel portal
ANGLE OF INCLINATION θ
F=6
F=8
F=9
F=10
F=11
F=12
F=10
F=8
ANGLE OF DIVERGENCE OF JET (ONE SIDE) IN DEGREES
TRANSITION BUCKET
FLAT-BOTTOM B
A. SPREADING OF JET
YAN HEE MODEL
FLAMING GORGE MODEL
WU SHEH MODEL
GLEN CANYON MODEL
HUNGRY HORSE MODEL
HUNGRY HORSE MODEL & PRO
FONTANA PROTOTYPE
ΔD2/D2 = REDUCTION IN T. W. DEPTH / TAILWATER DEPTH
A1/A2 = CROSS-SECTIONAL AREA OF RIVER ABOVE ZERO FLOW / CROSS-SECTIONAL AREA OF FLOW AT TUNNEL POR
B. TAILWATER DRAWDOWN

Fig. 9

Hungry Horse Spillway Tests. Spillway Discharge 30,000 cfs. Side View of Jet.



Hungry Horse Spillway Model—Discharge 35,000 cfs

Model-prototype Comparison

Fig. 10. Hungry Horse Spillway Flip Buckets

The intensity of the ejector action and the resultant lowering of t water at the bucket have been found to be a function of the energy in and the amount of resistance encountered when the jet strikes the ta In the curve of Fig. 9B the abscissa is the cross-sectional area of t flow near the point of impact of the jet divided by the cross-sectiona the flow at the tunnel portal. The river flow area is the product of t ence between the no-flow tail water elevation and the tail water elev the discharge being investigated, and the average width of the river point of impact. The area of the flow at the portal is obtained by div spillway discharge quantity by the average velocity. The ordinate is of the amount of drawdown to the depth of tail water. The depth of t is the same depth used to determine the river cross-sectional area.

The curve, defined by the test points shown, indicates with reaso racy the drawdown at each dam site for which data were available. points include various shapes and depths of channel and various type bucket jets. Further, the two prototype tests on Fontana and Hungry Dams showed good agreement between model and prototype test resu ever, in predicting drawdown at future sites, the curve should be us caution until more data are available.

Effect of Trajectory Shape

In addition to the effects of drawdown explained above, the jet tra important in other ways. The angle of the bucket lip with respect to determines the distance the water will be thrown downstream. Howe steeper the angle, the more the jet will be broken up and slowed dow resistance. Both of these effects cause the jet to enter the tail wate steeper angle. With a steep entry, the vertical component of velocit greater, and the jet will tend to dig into the channel bottom. With fl jectories, the horizontal component will be greater, the drawdown w

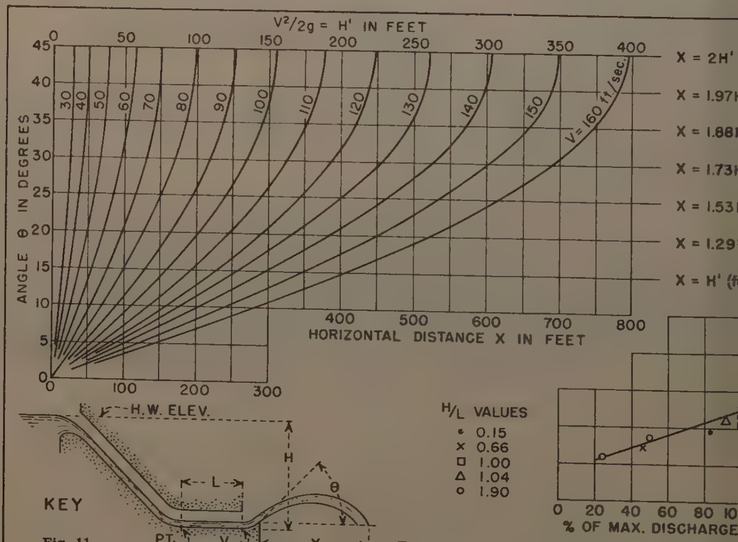


Fig. 11

TRAJECTORY LENGTHS AND HEAD LO

ter, and the forward velocity will be higher. High velocity channel flow persist downstream from the impingement area for a considerable ance if the channel bottom does not erode to produce a deep pool. High velty flow along the channel banks will then occur. If the bottom erodes, an rgy dissipating pool will be formed, and flow downstream will be smoother. ket flip angles are usually constructed from between 15° and 35°. Angles than 15° do not give enough lift to clear the bucket structure, and little is ally gained, from any viewpoint, by increasing the angle beyond 35°.

Fig. 11 contains a family of curves which may be used to estimate the traory length for inclination angles up to 45° and velocities up to 160 feet per ond. These curves were obtained from the simple equations for the path projectile, $X = V^2 \sin 2\theta/g$. For a given angle θ the equation may be simied as shown by the equations to the right of the trajectory curves. For 15°, X = H'; for θ = 45°, X = 2H' etc., where H' is the velocity head at the ket entrance. To estimate H', the curve in the lower right of Fig. 11 may used. Here, H', expressed as a percentage of the total head H, is plotted sus the percentage of maximum tunnel discharge. H' is seen to vary from ut 61 per cent for 20 per cent of maximum discharge, to about 75 per cent maximum discharge. Maximum discharge is considered to occur when the nel is about three-fourths full at the outlet portal. The points which demine the curve have ratios of vertical drop to horizontal tunnel length, L, from 0.15 to 1.9.

Trajectory lengths taken from these curves have been found to be reasony accurate when checked by hydraulic models. Some difference between del and prototype trajectory lengths may be expected to occur, however. tle is known regarding model and prototype trajectory length agreement, but m measurements estimated or scaled from photographs, and from actual asurements reported by the author,(1) it appears that the differences are ally not critical in nature. The prototype trajectory is shorter than the del or theoretical jet and has a steeper angle of entry into the tail water. e difference is believed to be caused by the greater air resistance enntered by the high velocity prototype jet. From sketchy information on a structures, the trajectory length in the prototype for 20 per cent of maxim discharge is believed to be 15 to 20 per cent shorter than in the model, . 12. There also are indications that the difference becomes less as the totype discharge increases.

In determining the radius of the bucket curve, it is necessary to provide a lius at least four times as great as the maximum depth of flow. This proes an incline sufficiently long to turn most of the water before it leaves the ket and provides assurance that the jet will be thrown into the desired area vnstream.

essures in the Transition Bucket

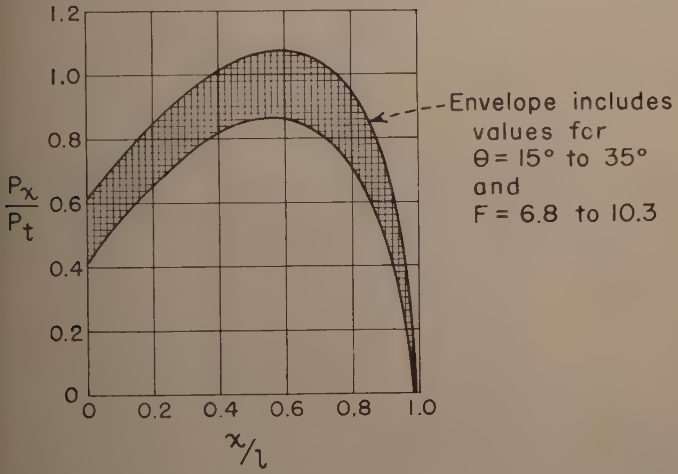
Because of the simplicity and effectiveness of the transition bucket, it will bably be used on many future tunnel spillways. Extensive pressure asurements were therefore made on several buckets having two different lination angles, 15° and 35°, to indicate that the buckets were safe against itation pressures and to provide data for structural design. The results hese tests have been summarized in Figs. 13, 14, and 15 and may be pful in making preliminary designs.

Both Prototype Tunnels Discharging 10,000 cfs Each

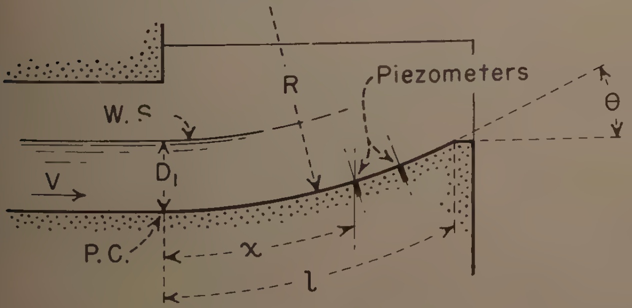
Both Model Tunnels Discharging 12,500 cfs Each

Fig. 12. Fontana Dam Spillway
Flip Buckets

Model-prototype Comparison



P_x = Measured pressure
P_t = Theoretical pressure; $(1.94\omega^2 R + 62.5) D_1$; where $\omega = V/R$
x = Developed distance from P.C. to piezometer
l = Developed distance from P.C. to end of bucket
F = Froude number, computed from V and D_1 at P.C.



SECTION ALONG ℄

PRESSURES ON
TRANSITION BUCKET FLOOR

Fig. 13

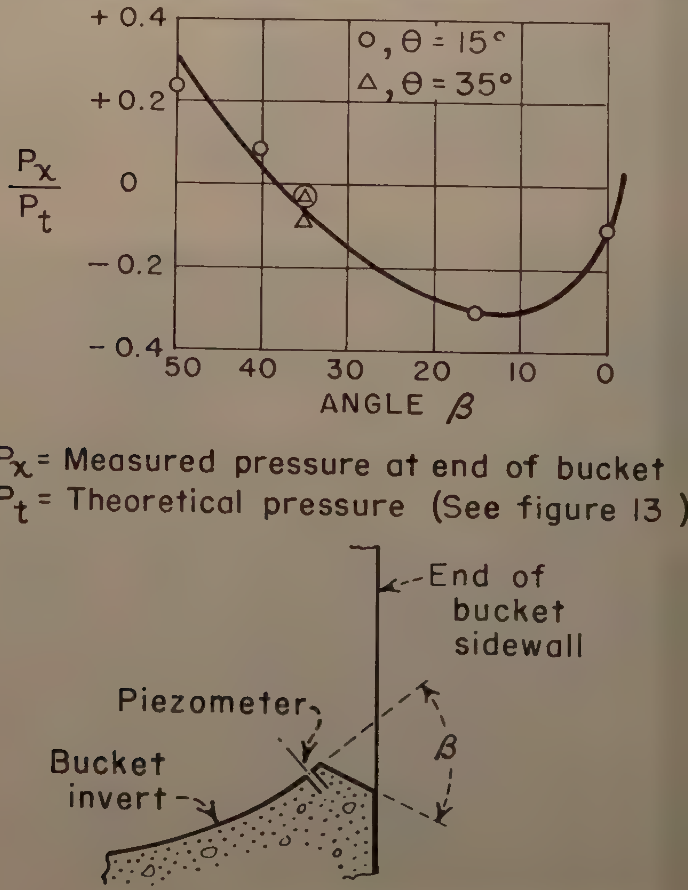
Fig. 13 shows pressures along the center line of the transition bu
floor. The envelope curve includes inclination angles from 15° to 35
flows in the Froude number range of 6.8 to 10.3, the usual range of
conditions. The maximum pressure was found to be slightly greater
en by Gumensky(2) from theoretical considerations. The theoretica
P_t, is expressed:

$$P_t = (1.94\ w^2R + 62.5)\ D_1$$

where

$$w = \frac{V}{R}$$

This maximum pressure occurred about 0.6 of the bucket length fro
stream end. Pressures rapidly became less toward the downstream
the bucket and reached atmospheric at the bucket lip.



PRESSURES AT END OF BUCKET

Fig. 14

or some tests a piezometer placed just upstream from the bucket lip, 14, indicated below atmospheric pressures, a phenomenon which has not satisfactorily explained. Experiments on model buckets showed that the sure on this piezometer was affected by the shape or angle of the down- am portion of the bucket lip. The curve of Fig. 14 shows the relation be- n pressure and the angle β. The curve indicates that for a given angle of nation θ, β should be 35° or more to insure atmospheric pressures or e at the lip piezometer. The curve also indicates that if β is 0° the pres- will be atmospheric. This is not a practical solution, however, since if 0° the piezometer will then be upstream from the lip and a new problem be created at the end of the extended bucket. It should be noted that the et side walls extend beyond the lip piezometer as shown in Fig. 14.

he curves of Fig. 15 indicate the pressures to be expected on the side s of the transition bucket from the base of the wall to the water surface. an inclination angle θ of 35° the maximum pressure is about 11 times as t as hydrostatic and occurs near the base of the wall at about the three- ters point, $x/1 = 0.75$, of the bucket length. At the end of the bucket, $= 0.99$ the maximum pressure is only four times as great as hydrostatic. $\theta = 15^\circ$ the maximum pressure is four times greater than hydrostatic at $= 0.26$, 0.55 and 0.80 and is only twice as great as static at $x/1 = 0.99$. er data are presented for different bucket radii, $R/1$ values, and stations g the bucket, $x/1$ values. Although the data are not complete, sufficient rmation is presented to make a preliminary structural design. On the ning Gorge Dam spillway bucket, one side wall was cut down to the spring of the tunnel without objectionable spreading of the jet occurring when the depth exceeded the height of the wall. This procedure simplified the ctural design of the bucket by reducing the overall load on a wall which no rock behind it.

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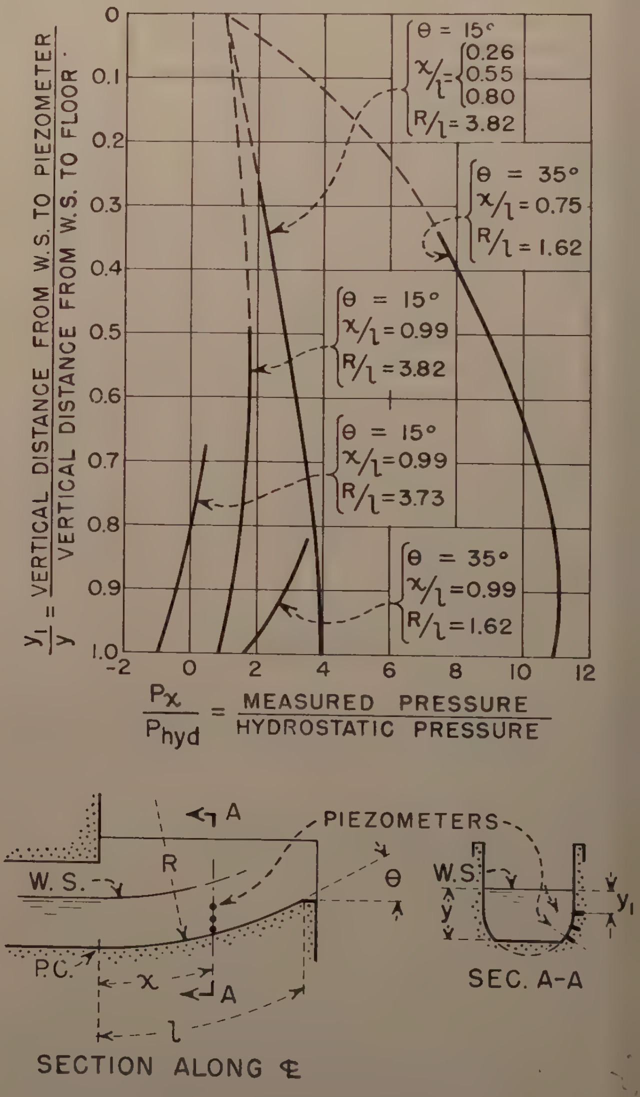

PRESSURE ON SIDEWALL OF TRANSITION BU

Fig. 15

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HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

TIDAL CHARACTERISTICS FROM HARMONIC CONSTANTS

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ABSTRACT

Harmonic analysis of tide observations determines harmonic constants plitudes and epochs) of the harmonic constituents of the tide. Formulas using the constants to obtain various ranges are furnished and techniques obtaining typical curves are illustrated for various ports. These are npared with predicted curves.

INTRODUCTION

Many engineers have an occupational interest in tides since tide observa-ıs provide a basic control in hydrography, are important relative to main-ance of rivers and harbors, and are a factor in pollution studies. In re-t years, projects involving storm-water levels and tidal-power develop-nt have emphasized the importance of tides to engineers. The tide tables nish adequate predictions for most engineering problems but there are ne aspects of tides that can best be generalized directly from the harmonic stants.

Harmonic constants are the amplitudes and epochs of the harmonic con-uents that make up the tide at any place. Through a modified Fourier lysis, using the periodicies of the movements of the moon and sun and sea-al meteorological variations, the observed hourly heights of the tide for eriod of observations are operated on to separate the tide into elementary rmonic constituents.

The determination of mean ranges, spring ranges (average near the times new and full moon), perigean ranges (average when the moon is closest to

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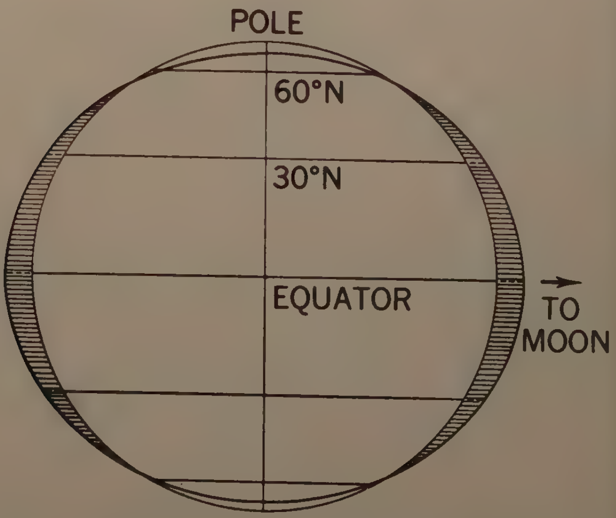
the earth), etc., and the typical shape of a tide curve are best deterr the harmonic constants. It is the purpose of this paper to indicate w monic constants represent and ways in which they can be used.

General Description

The tide is the periodic rise and fall of the water that results fro gravitational attraction of the moon and sun acting upon the rotating The tide-producing force, varying with the constantly changing posit both sun and moon relative to the earth, can be resolved into a numb simple periodic forces. To simulate the complicated motions of the sun, there are substituted a number of hypothetical tide-producing b each having a fixed period for its circular orbit around the earth in of the equator. Each hypothetical body produces a constituent tide w period that can be calculated accurately from astronomical data. Tl the actual tide at any place may be conceived as being made up of a of constituent tides, the period or speed of each being determined by propriate hypothetical tide-producing body.

Fig. 1 shows the theoretical tide on an earth completely covered layer of water when the moon is on the equator. Friction and inertia been disregarded so it is assumed that the waters have responded in the attraction of the moon. Note that there is a high water bulge on of the attracting body, another high water bulge on the opposite side, low water belt circling the earth in between, going from pole to pole istence of a bulge on the opposite side can be shown mathematically, the premise that the tide producing force at any point is the differen the gravitational attraction of the tide-producing body on a particle a



MOON ON EQUATOR

FIGURE I

t .nd the attraction on a particle at the center of the earth. In this mathe-ical development it is also shown that the tide-producing force varies in-sely as the cube of the distance, rather than as the square of the distance ıe gravitational attraction.(1)

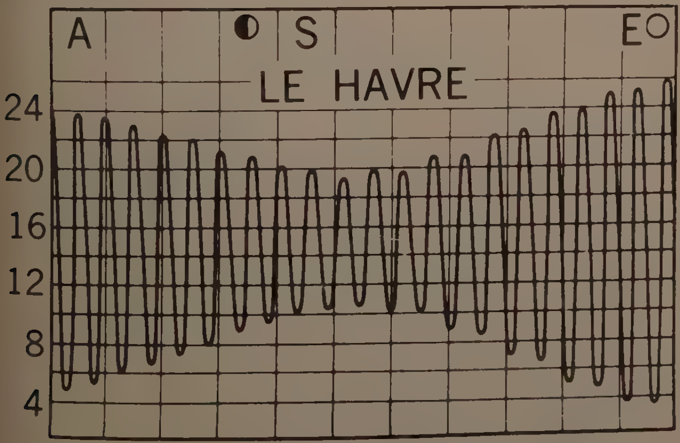
iod of Tidal Components

ınasmuch as the average interval between consecutive transits of the moon oer or lower) is 12 hours and 25 minutes, this is the period of the princi-lunar constituent. The component is designated by the symbol M_2, the M wing it is related to the moon's motion and the subscript showing that it is ıidiurnal (period of about half a day). The principal solar constituent is a ıponent named S_2 with a period of exactly 12 hours.

The components M_2 and S_2 will be in phase at both full moon and new moon. om Fig. 1 it is apparent that it will make little difference whether the moon sun are in line on the same side of the earth (new moon) or in line with center of the earth but on opposite sides (full moon). The large ranges ıew and full moon are called spring tides. With the moon at first or last rter, the M_2 high water coincides with the S_2 low water. This results in aller than average semidaily ranges of tide called neap tides.

The periods of M_2 and S_2 of 12.42 and 12 hours respectively (corresponding rly angular speeds of 28.98° and 30.00° per hour) will bring these com-ents in conjunction or opposition at the proper phases of the moon.

A simple example of a curve consisting predominately of M_2 and S_2 is wn in Fig. 2 for Le Havre, France. Note that at spring tides, with large ges, not only are high waters higher than average but low waters are lower n average. At neap tides, high waters are lower than average and low wa-s higher.



TIDE CURVE
FOR LE HAVRE, FRANCE

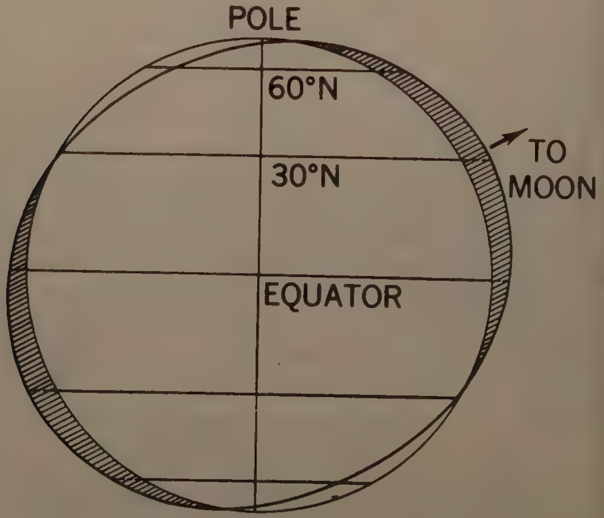
FIGURE 2

The moon moves in an elliptical orbit rather than a circular one. distance from the earth varies, its tide producing force varies acco being greatest when the moon is closest to the earth (221,000 miles and smallest when it is at maximum distance from the earth (253,00 at apogee). The period from perigee to next perigee averages about days. To simulate this changing attraction of the moon, we introduc ponent N_2 with a period of 12.66 hours (speed of 28.44° per hour) th in conjunction with M_2 at times of perigee and in opposition at time N_2 is known as the larger lunar elliptic semidiurnal component.

The distance of the earth from the sun also varies during the yea the degree of variation is considerably less than the moon from ear ponents representing this variation as well as numerous other lesse tions in the apparent paths of the moon and sun are used in tide pre but will not be defined in this paper.

Thus far we have dealt with semidiurnal tidal variations, restric moon and sun to the plane of the equator. However the declination o varies annually between about 23.452°S and 23.452°N while the moo treme declinations of 28.597°S and 28.597°N every 19 years.

Note in Fig. 3 what happens when the moon is at extreme north d The high water bulges are still present under the moon and on the o side. The low water belt still circles the earth in between, but no lo pole to pole. When the earth rotates on its axis, a point no longer e the same high tide with both upper and lower transits of the moon. latitudes there are two high waters of unequal height and at nother l just one high and one low water a day. Thus there is a diurnal vari pending on the declination of the tide producing body. The principal components introduced to simulate these variations are K_1 and O_1. combined luni-solar diurnal component with a period of 23.93 hours



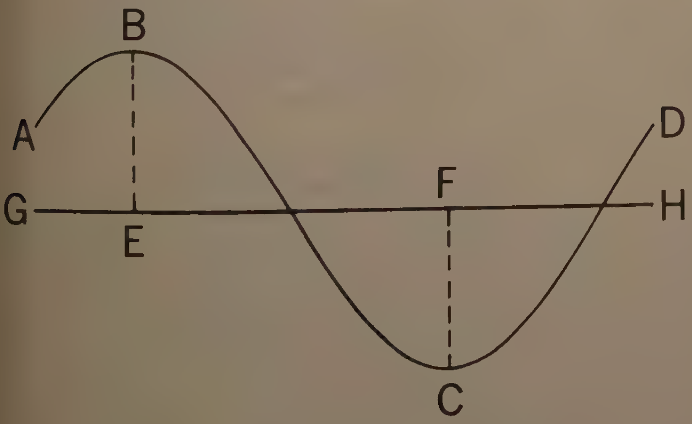
MOON AT MAXIMUM DECLINATION

FIGURE 3

la. speed of 15.04° per hour. O_1 is a lunar diurnal component with a od of 25.82 hours and an angular speed of 13.94° per hour. The sum of K_1 and O_1 angular speeds equals the M_2 angular speed, so that each day liurnal high water comes in the same position relative to the two semi-na! high waters. Furthermore the speeds of K_1 and O_1 are so chosen they are in conjunction when the moon is at extreme north or south dec-ion and in opposition when the moon is on the equator.

rmination of Tidal Characteristics

n the early part of this paper there was specified an earth with uniformly water and resulting instantaneous response to a tide-producing body. ever our earth does have land masses impeding the progression of tide es and the water areas are not uniformly deep. As a consequence the retical development gives us only the basic periods of the components of tide and some approximate relations between components but the ampli-s and epochs must be determined from observations by a special form of monic analysis.(1) Fig. 4 shows a complete period of a component tide. amplitude is the average semi-range, BE or CF, the vertical height from sturbed sea level, GH. If A is the time of meridian passage of a hypothe-l tide-producing body, the time from A to maximum high water B, is the ch or lag. It is expressed as an angle which is (GE ÷ GH) 360°.
Three systems of reference for the epoch of a component are used, each hich is designed for a particular purpose. Since the available constants be referred to any of these, each will be briefly mentioned. If A repre-s the time of passage of the hypothetical tide-producing body over the idian of the place for which tidal constants are furnished, the epoch is ked as "K" for Kappa. If A represents the time of passage over the



COMPLETE PERIOD OF COMPONENT TIDE

FIGURE 4

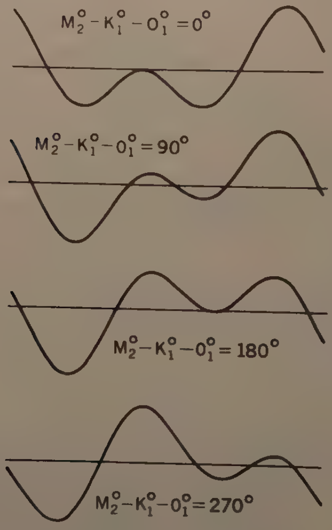
meridian of Greenwich, the epoch is noted as "G" for Greenwich tr
nally if A represents the time of passage over the meridian of the
ard time zone (for example 75° W for Eastern Standard time in Nev
the epoch is noted as a lower case "g" and the corresponding time
be indicated. Formulas for computing either K, G or g, given any
three, are furnished in Coast and Geodetic Survey publications.

Once an amplitude and epoch are determined for each componen
can be represented by a cosine curve. These curves are summed
on a tide-predicting machine and tide tables can be prepared for m
in advance provided that the related hydrography of water basins d
change. Ordinarily about 20 components are used for a station by
and Geodetic Survey although the machine is designed for 37 compo
Various combinations of components are used for different stations
components of significant amplitude, as obtained from analysis of t
vations, are used for each station predicted.

Examples of Combining Various Components

The amplitudes and epochs of the five components described pre
M_2, S_2, N_2, K_1, and O_1, can be combined to approximate a typical
curve, and to approximate various ranges and mean intervals.

As noted previously the combined hourly speed of K_1 and O_1 equ
hourly speed of M_2. Consequently, although the combined K_1 and C
will vary in amplitude depending on the declination of the moon, the
high water will always be fixed in position relative to the two semic
water of M_2. Fig. 5 shows some possible combinations of (K_1 + O_1
curves given equal amplitudes and various phase relationships of th
$M_2°$ -$K_1°$ -$O_1°$. The epochs may be Kappa, G or g, provided all a



COMBINED M_2 & $K_1 + O_1$ CURVES

FIGURE 5

e same meridian. Note that when $M_2° - K_1° - O_1° = O°$ the inequality is n the high waters, the low waters being equal. When the difference is the inequalities are equal and the sequence is higher high to lower low. a difference of 180°, the high waters are equal, the inequality being in ow waters. With a difference of 270°, the inequalities are again equal, he sequence is lower low to higher high. Using these four sets of curves uides, it is possible to visualize the relationship for any value of $M_2° - K_1°$ °. In combining the epochs, multiples of 360° may be added to arrive at lue between O° and 360°. Similar relationships will hold for M_2 and + O_1) amplitudes that are unequal; Fig. 5 may aid in visualizing various binations.

The amplitudes of the harmonic constants can be used to approximate var- average ranges. These have been derived from formulas and tables in st and Geodetic Survey Special Publication 260, Manual of Harmonic Con- t Reductions, 1952.

Mean range (long period average)	$= 2.2M_2$
Spring range (at new and full moon)	$= 2.1 (M_2 + S_2)$
Neap range (at quadrature)	$= 2.1 (M_2 - S_2)$
Perigean range (moon closest to earth)	$= 2.2 (M_2 + N_2)$
Apogean range (moon farthest from earth)	$= 2.2M_2 - 1.7N_2$

When M_2 is small relative to K_1 and O_1,
Tropic range (diurnal range at extreme declination of moon) $= 2(K_1 + O_1)$

f the epochs of the various components that combine to form spring tides, igean tides, etc., were equal, these particular ranges would occur exactly ew and full moon, perigee, etc. This is not ordinarily the case, and the can be calculated exactly from the epochs. The lag in hours for spring neap tides is found by the formular $0.984 (S_2° - M_2°)$, the lag in perigean apogean tides, by the formula $1.837 (M_2° - N_2°)$; and the lag in tropic s by $0.911 (K_1° - O_1°)$. A positive lag means that the tide follows the ap- priate astronomical event; a negative lag indicates that the tide precedes event.

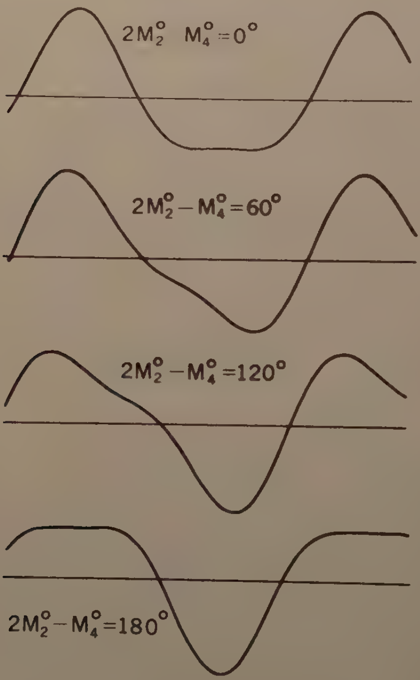
uence of Tidal Basin on Tidal Characteristics

Some tidal components of significance are not astronomical in nature. M_4 M_6 are harmonics of M_2, their angular speeds being exactly two and three es that of M_2. These are called shallow water components and reflect the ortion of the cosine-type tide curve because of the physical features of ain basins. The period of a basin is determined by its length and depth. n the period of a basin approximates a harmonic of the tidal period, it in- luces a repetitive disturbing wave that more or less uniformly distorts the pe of the tidal wave. Figs. 6 and 7 show the shape of the curve as affected M_4 and M_6 respectively. The degree of distortion depends on the ratio $/M_2$ or M_6/M_2 and the portions of the curve principally affected depends he phase difference $(2M_2° - M_4°)$ or $(3M_2° - M_6°)$.

eorological Effects

Finally there are variations in mean sea level due to periodic meteorolog- forces. Seasonal changes in prevailing winds have a measurable effect

on monthly mean sea level. Sa and Ssa are two components designed
one cycle and two cycles respectively each year. The annual sea lev
can be approximated by a combination of these two components whic
effect on the times of tides because they change so slowly. Because
slow rate of angular change for these components, approximate resu
obtainable disregarding whether the epochs are Kappa, G or g. The
Sa divided by 30 denotes the maximum point of the annual variation i
level in months after the vernal equinox (about March 21). Thus an
40 ° represents a maximum on about May 1. The epoch of Ssa is div
60 to obtain a semi-annual peak in months after the vernal equinox.
peak follows six months later. Thus an Ssa epoch of 140 ° represent
mum sea level peaks on about June 1 and December 1. Sa and Ssa a
determined from a number of years of observation. The published v
some places have been based on one year's observations; when this
case, they may be unreliable.



SHAPE OF CURVE AS AFFECTED BY M_4

$M_4/M_2 = 0.3$

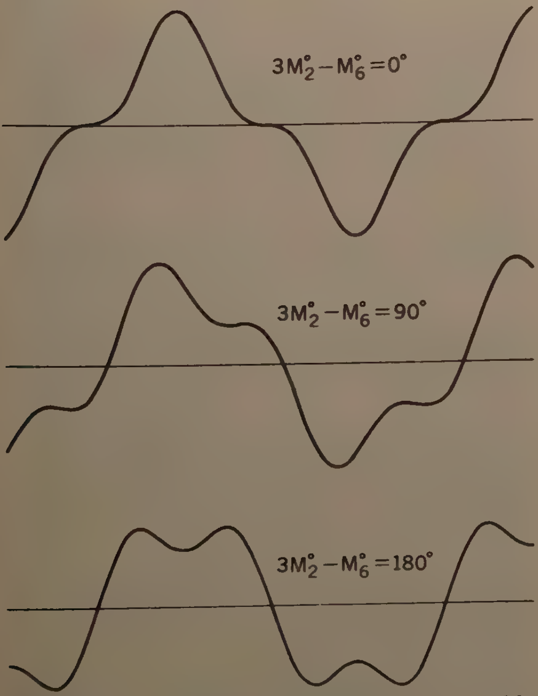
FIGURE 6

SUMMARY

me of the more important techniques can be demonstrated using the har- : constants listed below and comparing the relationships with curves in 8 and 9, taken from the Coast and Geodetic Survey East Coast and West Tide Tables. The symbols at the top of these figures represent from) right, apogee, last quarter, on equator, and new moon.

Principal Diurnal and Semidiurnal
Harmonic Constants for Various Places

	K_1		O_1		M_2		S_2		N_2	
	H feet	G deg.	H feet	G deg.	H feet	G deg.	H feet	G deg.	H feet	G deg.
on	.46	206	.36	187	4.48	111	.74	146	.96	80
acola	.41	58	.41	49	.06	172	.02	179	.01	54
le	2.73	278	1.50	256	3.53	12	.84	37	.71	341
orage	2.22	340	1.17	324	11.56	106	3.16	149	1.97	78



SHAPE OF CURVE AS AFFECTED BY M_6
M_6/M_2 1/3
FIGURE 7

For Boston the most obvious relationship is the large amplitude for M_2 compared to K_1 and O_1. This indicates relatively little diurnal inequality a a large semidaily range, as is brought out by the curves. Somewhat peculia to the New England coast, N_2 is larger than S_2, causing a larger mean perigean range than the mean spring range.

At Pensacola we have a completely different tidal regime. Although K_1 a O_1 are small, the semidaily components are relatively negligible. This results in a small diurnal tide that becomes negligible when the moon is on th equator because K_1 and O_1 oppose each other at that time.

At Seattle both daily and semidaily components are large and we have a substantial diurnal inequality. Combining epochs, $M_2° - K_1° - O_1° = 198°$ (2 x 360° have been added to obtain a positive value). Going back to Fig. 5, we have a value between 180° and 270°, but considerably closer to the lowe value. Thus we expect a high water inequality substantially smaller than th low water inequality and a sequence of lower low to higher high to higher lo to lower high. Looking at the curve in Fig. 9 we find the above except durin a transitional period when the moon is near the equator.

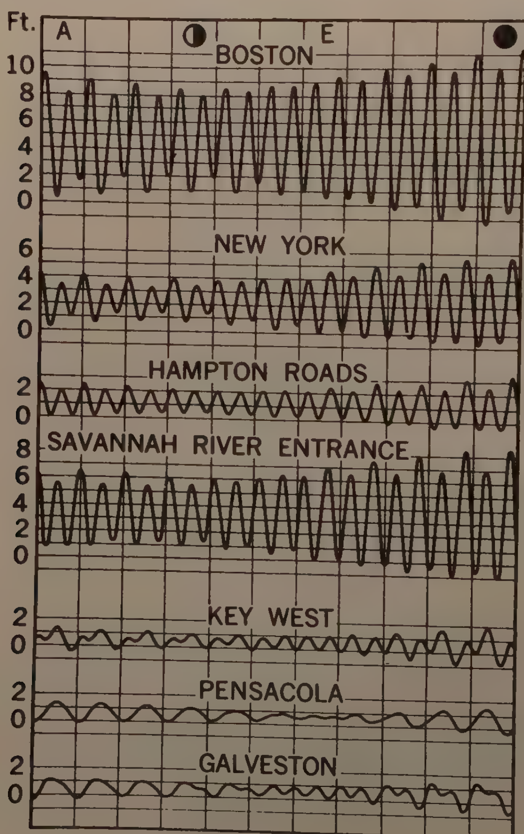


FIGURE 8

Finally at Anchorage the outstanding characteristic is the large amplitudes of the semidaily components. It follows that there is a large semidaily range with relatively small inequality and, more typically than at Boston, larger spring ranges than perigean ranges. Since $M_2° - K_1° - O_1° = 162°$, the inequality, although small, is larger in the low waters than the high waters. Fig. 9 bears this out.

The above are illustrated as examples of how data on typical shapes of the tide curves can be readily estimated from the harmonic constants listed. Other deductions on shape of curve can be made from the harmonic constants M_4 and M_6, and of seasonal variation in sea level from the components Sa and Ssa.

REFERENCE

1. Coast and Geodetic Survey Special Publication No. 98, Manual of Harmonic Analysis and Prediction of Tides by Paul Schureman, 1941.

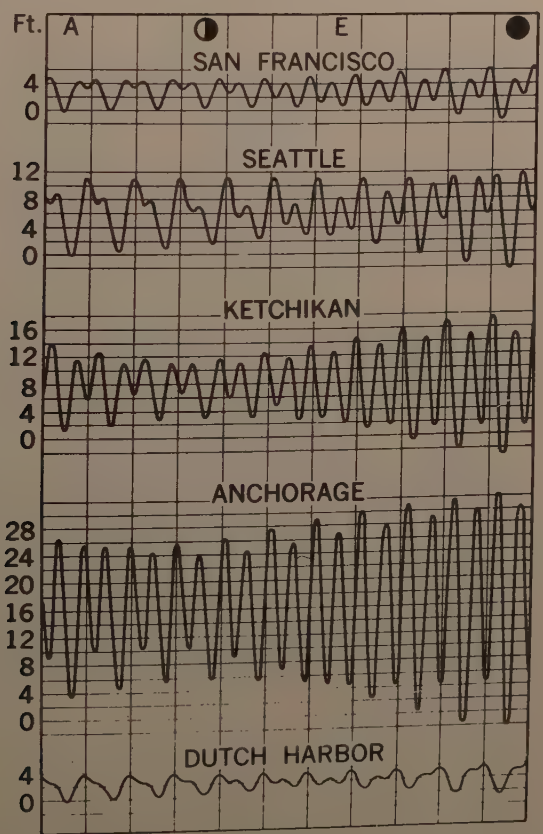


FIGURE 9

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CONTENTS

DISCUSSION

(Over)

Note: Paper 2271 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, HY 12, December, 1959.

INTERIM CONSIDERATION OF THE COLUMBIA RIVER ENTRANCE[a]

Closure by John B. Lockett

JOHN B. LOCKETT,[1] F. ASCE.—The author wishes to thank Mr. R. E. Hickson for his discussion. During preparation of the paper, Mr. Hickson was furnished a copy of the final draft for his information and he took that opportunity to furnish his comments thereon. His published comments, although they are in substantial disagreement with the views expressed by the author, are welcomed since honest, constructive criticism is believed to be one of the keystones of human progress.

The paper points out that conditions at the Columbia River entrance have been undergoing and are still continuing to undergo a process of change under the influence of completed corrective and training works. Completion of the original South Jetty in 1895 was followed by extension of that structure in 1913, construction of the North Jetty in 1917, and construction of Jetty "A" in 1939, all of which were provided under the 30 and 40-foot entrance projects. This, followed by extensive dredging under the 48-foot project adopted in 1954, has subjected the entrance area to various degrees of control, resulting in continuously changing conditions within the entrance as river and ocean forces attempt to reach a state of equilibrium. Hence, within the time of permanent improvement, it is not believed logical to select any one short period in its history as representing an optimum stage of development. It is, therefore, believed that it may never be possible to restore entrance conditions to those prevailing during the period 1916-32, mentioned as a goal to be achieved. Instead, it is believed that we must work with conditions as they exist today and then determine how these conditions may be further improved for tomorrow.

Without casting discredit on the engineers responsible for the design and construction of existing Columbia River entrance structures, who certainly performed miracles of accomplishment considering the prevailing conditions and the limited resources available, it must be admitted that practically all of this work was, of necessity, planned, designed, and constructed largely on the basis of judgment considerations. Some of this work has been extremely successful while the true effectiveness of other work is still in doubt. Accordingly, it was considered advisable in connection with the adoption of the 48-foot entrance project, which includes provisions for construction of spur Jetty "B", that further work, including dredging, be based on known engineering fact rather than on judgment considerations as governed in the past. In view of the rapid advances made in the field of tidal hydraulics during the past 10 years elsewhere in the nation, it followed that a new approach to the problem at the

a. Proc. Paper 1902, January, 1959, by John B. Lockett.

1. Head, Special Projects Investigation Section, Planning and Reports Branch, Eng. Div., U. S. A. Engr. Div., North Pacific, Portland, Ore.

Columbia River entrance was indicated and that this new approach should include a searching engineering study of all possible phenomena which might have an important bearing on the problem. Mr. Hickson's position in this regard, however, may be summarized by the following extracts from his initial comments on the final draft of the paper:

> ". . . it appears we are making a very complicated problem out of one which is relatively simple. It is simply a problem in river hydraulics in which the principles of hydraulics apply. It differs from inland river control only to the extent that winds, waves, tidal currents, density currents, and littoral currents distract our attention from the basic consideration, which is that the river outflow will produce and maintain a satisfactory channel if properly directed as to alignment and width between controlling bank or constructed works. This has very successfully been done on the upriver ship channel of the Columbia and other streams. . . . It appears that the problem need not be confused, or a solution made more difficult by attempts to make a detailed analysis of all the forces which are at work and what part each may have in producing or changing conditions. . . ."

This concept raises a question as to how we may properly direct the river outflow to produce and maintain a satisfactory entrance channel when we know neither the full nature nor magnitude of the forces controlling the regimen of the estuary, let alone the effects of these forces on any further regulatory structure which may be devised.

The author, in general, does not believe that any strong exception could be taken to the discussion provided it was based on conditions prevailing prior to improvement at the Columbia River entrance when the forces of nature were such as to prevent saline intrusion into the estuary. The subsequent construction of corrective works and dredging, has, however, upset the balance of natural forces and has allowed the penetration of saline influences to the point where they are believed to represent the major dominant force controlling the regimen of the estuary today. This is a logical sequence of events which has been confirmed by histories of estuary improvement elsewhere.

There are, however, several points brought out by the discusser which relate quite fundamentally to the principles of tidal hydraulics and should, therefore, be considered in further detail. First of all, it is not believed that proper design of any estuary structure can ignore saline influences as represented by density currents. These currents are created by differences in specific gravity of the fresh river water and the salt water of the sea. The magnitude of these currents is difficult to fully describe but may be visualized to some extent if we consider the hydrostatic conditions created by the meeting of the fresh river water weighing about 62.4 pounds per cubic foot with the salt water of the sea weighing about 64.0 pounds per cubic foot. The difference in specific gravity, reflected by the difference in weight of the sea and fresh water or 1.6 pounds per foot of depth, creates a hydrostatic head in an upstream direction with force of about 80 pounds per square foot at the 50-foot depth level. This is equivalent to a total resultant force of over 5,000,000 pounds when applied through the authorized one-half mile channel width at the Columbia River entrance. The true magnitude of this force becomes somewhat greater than 5,000,000 pounds when we consider that the full width between the Columbia jetties is at least two miles. This force, evidenced by density currents, acts throughout the region of saline penetration within the

estuary and is kept in balance by counteracting forces of friction along the estuary bottom and inertial energy of flowing fresh water. Until this density current force is finally dissipated at the upstream limit of saline penetration, it remains a factor deserving of detailed consideration in the planning of estuary structures. Therefore, the author cannot agree with the concept that we should not allow saline influences to distract our attention in the consideration of this problem.

Considerable emphasis has been placed in the discussion on the interpreted results of the 1932 current survey which was undertaken along a single range across the estuary about five miles above the outer ends of the Columbia jetties. Equipment employed to measure current velocities at different depths at each of the five measurement stations along this range consisted of the ordinary Price current meter suspended from the metering vessels. Facilities to determine or measure the direction of subsurface currents were, of course, not available at that time and, as a consequence, the actual direction of subsurface currents was largely a matter of personal judgment. Further, the survey did not include concurrent measurements of salinity and temperature along this range and, hence, it was not possible to confirm the judgment decisions of current direction with measured indications of salinity. It is regrettable that a substantial part of the detailed field records made during this current survey has been lost over the years and, accordingly, its value has been reduced by circumstances beyond control. Even though all these observations were available today, it is not believed, due to changes occurring within the interim period, that they could be accepted as representative of conditions presently prevailing in the estuary.

In this connection, the discusser states that, in general, the same littoral and density currents prevailed in the entrance during the period 1916 to 1932 as prevail today. Although the magnitude of the littoral effect is still unmeasured and, therefore, the accuracy of this part of the statement can neither be confirmed nor denied, there is logic to the belief that the density current pattern is substantially stronger today than it was during 1916-1932. Examination of condition surveys undertaken during that period reveal that controlling depths in the entrance, except between 1925 and 1927, were less than 44 feet and in 1916 were only 36 feet. In 1925 and 1927, controlling depths were 48 and 49 feet, respectively. It is a recognized principle of tidal hydraulics that when depths of estuarine channels are increased for navigation or other purposes, the effective density head at the entrance is increased and density current action within the estuary becomes more pronounced. Since 1932, the controlling depth between the jetties has been increased by at least 6 feet and it is, therefore, logical to expect greater density current action now than during the referenced period. Even though the channel section were held constant by the construction of dikes or contractive works, such as the proposed Jetty "B", deepening of the entrance channel must increase density current effects in the saline region of the estuary.

The discusser also states that salinity measurements through a tidal cycle show that most of the salt water is washed out by each ebb tide. He refers to Fig. 3 of his discussion showing in graph form measurements of specific gravity taken opposite Fort Stevens, Oregon, near the Point Adams Coast Guard Station, river mile 7.5, in 1936 as support for this conclusion. Although it is known that actual density of water varies somewhat with its temperature, there appears to be no record of concurrent temperature measurements taken at that time. Therefore, in the absence of temperature data, it is doubted that

these observations can be considered conclusive. In any event, it is difficu to reconcile these observations with those made at practically the same location in 1955 by a disinterested agency as reported in Table 4 of the paper under discussion. It is noted that, in forwarding his report on the 1936 obs vations to the Committee on Tidal Hydraulics, Corps of Engineers, Mr. Hic son, in his letter memorandum of April 10, 1953, stated that ". . . It is realized that the basic data secured were not comprehensive enough to admit drawing firm conclusions". Some six years later, however, these same ob vations are presented as firm data warranting the conclusion that ebb force remove most of the saline influences from the entire estuary. No work has been undertaken since 1953 which might lessen the magnitude of saline influ ences in the Columbia River estuary and there appears to be no basis for a appraisal of the value of the 1936 observations. Even though these particul observations were of such nature as to permit the formation of a firm conclusion, such a conclusion could only apply to conditions prevailing in the v cinity of Fort Stevens, river mile 7.5, but could not be indicative of conditi occurring elsewhere in the estuary.

The discusser also stresses the need for early construction of spur Jett "B" as a regulatory structure designed to reduce dredging and provide a st channel through the entrance. He presents cross-sections of the estuary in the general vicinity of this proposed structure and indicates thereon the approximate flow area that would be cut off by the structure. He is of the opi that Jetty "B", by denying the river use of the area thus cut off, would contract the available cross-sectional area to the point where increased river velocities would solve the shoaling problem by erosion of Clatsop Spit shoal along the opposite shore.

Examination of the accompanying Fig. 1 shows that proposed Jetty "B" would lie along the north shore of the estuary, almost parallel to the North Jetty, with its extreme end approximately bisecting the distance between th ends of Jetty "A" and the North Jetty. Fig. 2 indicates that Jetty "B" would constructed in depths averaging less than 25 feet below MLLW and in a ma mum depth of 40 feet at its southernmost extremity. Of a total cross-secti area of about 521,000 square feet, Jetty "B" would occupy about 160,000 sq feet, or roughly 31 per cent of the total section. Under conditions of norma river flow, where there are no saline influences or reversals of currents t distract our attention, such a reduction in section could be expected to prod some effect on the direction of river flow by increasing current velocities t the point where they might erode less firm areas along the opposite shore.

Considering, however, flow conditions as they are believed to exist in th part of the Columbia River estuary, the prospect of solving the problem by this relatively simple expedient becomes somewhat less than promising. T is due to the fact that reversals of flow, created by tidal action, introduce factors that complicate or nullify the application of the principles of pure ri hydraulics. Under such conditions, it may be seen that the confining effect Jetty "B" would be limited to that portion of the time in which the entire flo throughout all depths in this restricted cul-de-sac area, as well as in the m Columbia channel to the south, would be in an ebb direction. It is believed that this condition, however, could be expected to prevail for only relatively short periods of time due to bottom flood flows generated by density curren action, even during an ebb tide. During that portion of the time in which the flow in the cul-de-sac would be in a flood direction, it appears that Jetty "B would be unable to affect the critical portion of Clatsop Spit lying to the wes

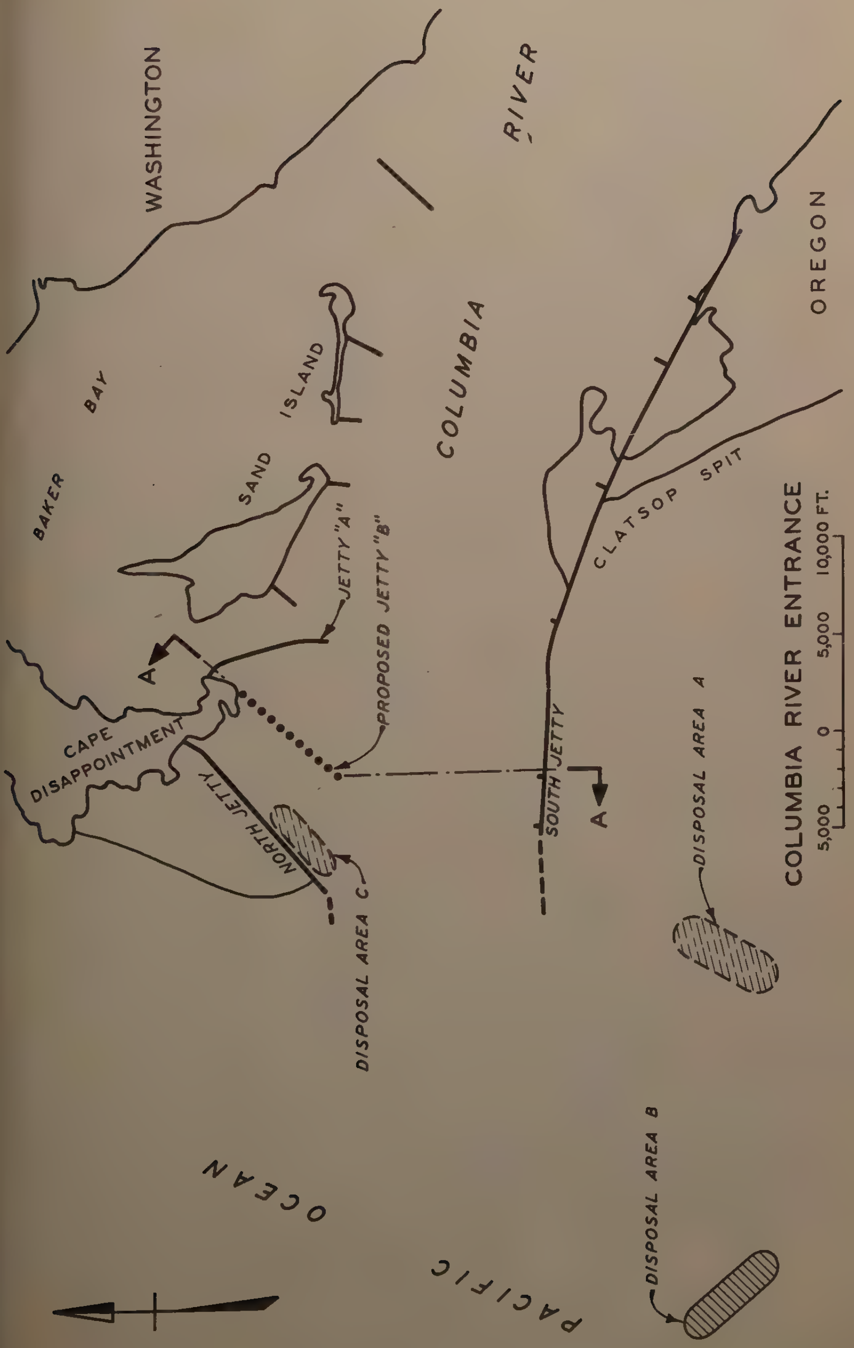
WASHINGTON
OREGON
COLUMBIA RIVER
BAKER BAY
SAND ISLAND
CAPE DISAPPOINTMENT
JETTY "A"
PROPOSED JETTY "B"
NORTH JETTY
SOUTH JETTY
CLATSOP SPIT
DISPOSAL AREA A
DISPOSAL AREA B
DISPOSAL AREA C
PACIFIC OCEAN
A
A
5,000 0 5,000 10,000 FT.
COLUMBIA RIVER ENTRANCE

FIGURE 1

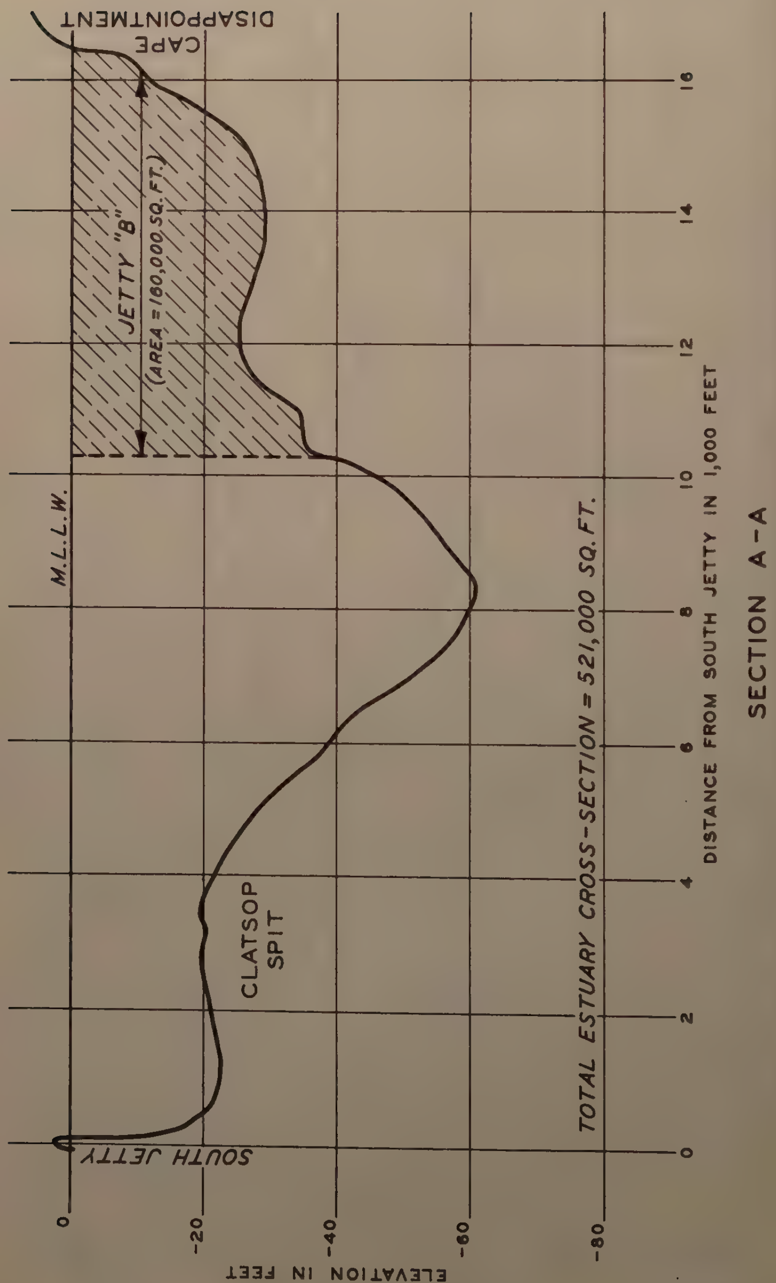
CAPE
DISAPPOINTMENT
JETTY "B"
(AREA = 160,000 SQ.FT.)
M.L.L.W.
CLATSOP
SPIT
SOUTH JETTY
TOTAL ESTUARY CROSS-SECTION = 521,000 SQ.FT.
0
-20
-40
-60
-80
ELEVATION IN FEET
0
2
4
6
8
10
12
14
16
DISTANCE FROM SOUTH JETTY IN 1,000 FEET
SECTION A-A

FIGURE 2

The structure might, instead, create eddy conditions which could conceivably threaten the stability of the North Jetty. Further, the main pattern of flow in this portion of the estuary would continue unaltered in the area immediately south of the structure where depths extend to 60 or more feet. It is believed that any density current action occurring in that principal flow area would, therefore, be unaffected by Jetty "B". Hence, there appears to exist reasonable basis for doubt regarding the true value of that structure.

The discusser claims that construction of Jetty "A" in 1939 stabilized the channel at that point, caused a widening of the channel for 800 feet to the south, and presently maintains depths in excess of 50 feet for a distance of over a mile downstream. Although the author does not disagree with evidence showing that the 40-foot contour opposite Jetty "A" moved southward following construction of that structure, it must be pointed out that this local improvement was accompanied by deteriorating conditions at the northwestern portion of Clatsop Spit, where the 40-foot contour in that area advanced markedly toward the north and west. Also, if we examine condition surveys prior to 1939, it will be noted that depths considerably in excess of 50 feet have prevailed for over a mile downstream from this point ever since improvement of the entrance was initiated. Accordingly, it is not clear how Jetty "A" may properly be credited with maintaining depths which have always existed in that area.

In regard to the effectiveness of Jetty "A" and the dikes along Sand Island, and the need for comprehensive study of the problem, the Committee on Tidal Hydraulics, in its reply of June 12, 1958 to Mr. Hickson's comments on Committee Technical Bulletin No. 2, which are both listed as unpublished references to the discussion, stated:

> ". . . The second exception concerns your statement that the spur dikes along Sand Island and Jetty "A" have maintained an excellent channel since their completion about 1938; the Committee does not share your view that these structures have been as effective as is claimed. The Committee feels very strongly that the recommended program of prototype measurements and the model study of the Columbia River entrance should be carried out at the earliest practicable date. Until these missions have been accomplished, and the results obtained have been fully analyzed and evaluated, the Committee believes that the construction of any so-called improvement works in the problem area would be a grave mistake. The existence of such wide differences of opinion as to the basic reasons for this shoaling problem, among engineers who have devoted most if not all of their careers to the study of tidal hydraulics and related matters, is ample proof to the Committee that this problem is worthy of careful and complete study and evaluation prior to the expenditure of large sums for improvements".

Fortunately, the plan of action recommended by the Committee was initiated by Portland District with the completion in 1959 of a prototype measurement program which observed current velocities and directions as well as salinities and temperatures at some 23 stations along six ranges across the estuary. Observations of these phenomena were made at five depth levels on 30 minute intervals at each station throughout complete tidal cycles of 25 hours each under conditions of low, normal, and high river discharge. The observed data are currently being evaluated and the author hopes to present some of the findings of the program in a forthcoming paper.

Preliminary to the conduct of this measurement program, Portland Dis made several tests of current velocity and direction equipment offshore at Columbia River entrance. As it had been suspected that dredge spoil disp in Areas A and C, which had been used for this purpose for many years, fo its way back into the entrance channel, tests were made in the vicinity of t particular spoil disposal areas. These tests revealed that such was quite likely the case and, beginning in 1958, Areas A and C were abandoned and practically all spoil was disposed in Area B, in depths of 120 feet or more This change in disposal practices is considered by the author to be respon ble for the securing in 1958 and 1959 of improved entrance depths by the e of each dredging season at substantially reduced dredging costs. It is beli that through this single expedient, the savings in dredging costs thus obtai will repay in a few years, the entire cost of the Columbia River tidal hydr investigation.

The author recognizes that it is quite probable that final evaluation of t prototype measurement program may not completely confirm all his views garding the behavior of currents and other phenomena at the Columbia Riv entrance. That is of little importance. The important thing is, of course, fact that positive action is now being taken to learn specific details regard the forces controlling the regimen of the Columbia River estuary so that l cal and factual planning can precede actual design and construction of any further regulating structure or structures that may be found necessary.

Adoption of modified spoil disposal practices at the Columbia River entrance, based on modern technological knowledge in the field of tidal hydraulics, has already been rewarding and the continued application of this knowledge to the over-all problem holds the promise of still greater rewa in the future.

HYDRAULIC ANALYSIS OF SURGE TANKS BY DIGITAL COMPUTER[a]

Discussion by John P. Herak and Salvador Rodriguez

JOHN P. HERAK[1] and SALVADOR RODRIGUEZ,[2] A. M. ASCE.—The author has made an important and significant contribution to the literature of hydraulic transients. The writers know of no other contribution opening such large avenues to refinements in the treatment of transients and to eliminating the usually tedious labor of making arithmetic integrations. Two major points impress the writers. The first is the close check given to the manual computations for all large maneuvers. This also emphasizes again the inherent safety resulting from the use of orifices or equivalent restrictions. The second is the opportunity for making refinements in the solutions, thus producing mathematical results for many minor factors that heretofore were necessarily intelligent guesses, unless an undue amount of labor were to be expended. On the other hand this requires a closer look at the assumptions and coefficients used for these minor effects, in order for them not to be misleading. It is hoped that Yu-Tek Li's researches, referred to later, can be published soon, since they indicate considerable variation from some of the results, such as those of Vogel and McNown, previously available.

The following is an evaluation of the differences between manual and machine results of the surge tank stability study.

The evaluation of tee losses, in the arithmetic integration study, were based upon values obtained using Vogel's data for shape 2 (rounded edges) of tee connections, and a branch to main diameter ratio of 0.58. Tee loss coefficients prepared by the Omaha District, C.E., (Table 5) from Vogel's data were based on a branch to main diameter ratio of 0.667 and at zero discharge ratio have been set equal to zero, thereby modifying Vogel's results in the range of zero to 0.1 discharge ratios. The effects of these differences between manual and machine coefficients are noted below.

The arithmetic integration method of investigating the stability of the Oahe System was to subject the unit to a relatively small instantaneous load change and determine whether the amplitudes of the tank levels with respect to the new steady level were damped or magnified with increasing time. As stated by the author, manual stability computations proceeded directly from point M_1 to point M_2 (Fig. 11) and once entry onto the curve Bhp_2 was made, constant power was assumed to be delivered by the turbine up to full gate position. This transient produced oscillations in Q_5 and H along the constant Bhp_2 curve of 78000 without at any time reaching full gate. The arithmetic integration neglected the effects of governor dead time, friction losses, inertia, and

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a. Proc. Paper 1996, April, 1959, by Nicholas L. Barbarossa.

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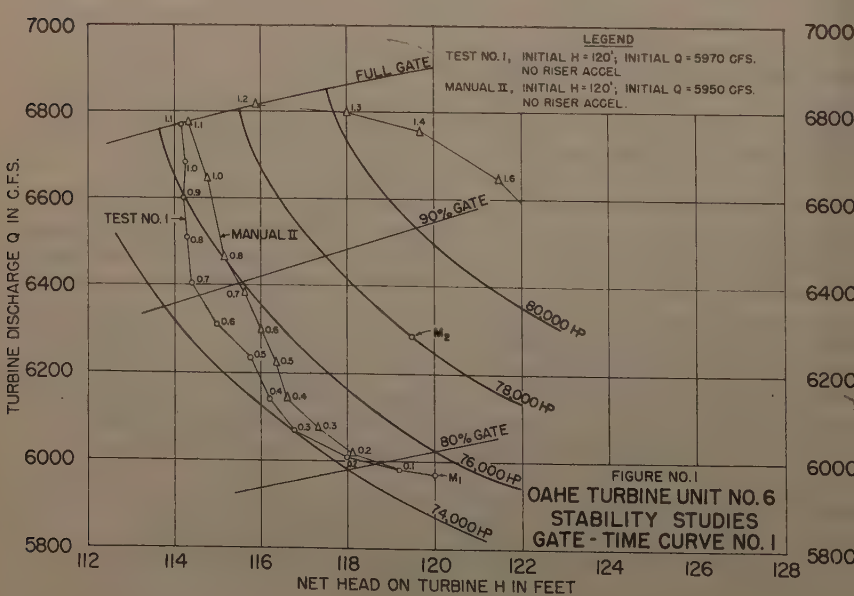
2. Senior Engr. II, Sverdrup & Parcel Eng. Co., San Francisco, Calif.

elastic energy in the hydraulic system downstream of the surge tanks; and energy storage and inertia of the rotating mass and electrical system. The computer studies included only partial corrections for some of these factor and did not consider the others. However, a step in the right direction was made by including the partial corrections.

A partial arithmetic integration study was made of Test No. 1, Fig. 11, ing a time interval of 0.1 second and the results are shown on Fig. 1 as Ma al II. The transient path to full gate position is similar to that obtained by computer, the variation apparently being due to differences in values of tee loss coefficients. In both cases, the transient attains full gate position befo reaching the Bph_2 curve. We believe the computer should have been programmed to permit the transient to follow the full gate curve, as indicated Manual II study to intersection with Bhp_2, with some overshoot due to gover nor dead time and system inertia. (Dead time considered equal to 0.05 sec for Manual Study II.) At this point the governor takes over and begins clos the turbine gates since the turbine output exceeds the required horsepower. The path of the transient will approximate a spiral and will converge on the new steady state point, if the unit operation is stable. It is suggested that t computer be programmed as outlined above and allowed to traverse throug several cycles to obtain a better measure of both surge tank and unit opera stability. It would be of interest to compare the decrement Δ (Jaeger), a measure of the damping of the hydraulic system, obtained by machine meth with that obtained by the manual method.

The transient curves on Fig. 12 all indicate that the gates begin to close before the new steady state power Bhp_2 is attained. This is not compatible with governor action.

As previously noted, differences in tee loss coefficients existed between manual and machine computations. A recent unpublished paper entitled "H Loss in Tee Section and Manifold",(1) by Yu-Tek Li, February 1959, a forn



member of our hydraulic section connected with the Oahe Reservoir Project, presents values of head losses at tee sections, head losses through orifices in the case where the plate orifice is installed in the lateral pipe, including values for different shapes of orifice plates, and for different locations of the orifice plate in the lateral pipe; together with a determination of head losses in a manifold system. Attention is being called to the paper, not with the intent of defending either set of tee loss coefficients used in these studies, but with the thought that while the writers agree with the author's ideas on the solution of transient problems by computer, additional basic work is still required in evaluating items such as head losses at tees before "exact solutions" can be made with computers.

In conclusion, the writers wish to commend the author on his excellent paper and look forward to the publication of his paper "Proposal for Comprehensive Study by Digital Computer of Hydraulic and Governing Transients Affecting Design and Operation of a Power Plant System".

REFERENCE

1. A dissertation presented to The Imperial College of Science and Technology London, England, 1959.

RESISTANCE EXPERIMENTS IN A TRIANGULAR CHANNEL[a]

Discussion by J. van Malde

J. van MALDE.[1]—This paper deserves study as the experiments described are unique regarding the sizes of the flume used, whilst the accuracy with which they were carried out make the results most valuable. The paper affirms that an open channel cannot be calculated with the well-known formulas of Prandtl (which were found from experiments with circular pipes) without making an error. According to their interpretation of the data the authors state that the average error in the mean velocity when using these formulas would amount to + 3,5% for the observed triangular channel being a smooth flume and - 2,2% in the case of the channel being a rough flume; these numbers refer to tranquil, non-critical flow. The authors conclude that in both cases the logarithmic formulas of Prandtl have no advantage over the Manning formula and that in the second case the formula of Morris for isolated-roughness flow has no advantage over Manning's or Prandtl's formulas either.

By studying the data for the hydraulic rough flume (runs 49-101) it appeared that the authors put together all values of ψ (the relation between f_o observed and f_p for smooth pipe) for a certain roughness, irrespective of slope (S) and depth (y_n). O. Kirschmer, who as far as the writer knows first pointed to the resemblance in resistance of a hydraulic rough open channel to a hydraulic smooth circular pipe(1) mentions in one of his papers(2) the possibility that ψ is influenced by S. This he concludes from some of the experiments of Bazin, taken in a rectangular flume with battens fastened to the walls in the direction of the flow; ψ would increase with increasing S. Theoretically there is no explanation for this influence, but as the theory for the uniform steady flow in open channels still doesn't take into consideration all the phenomena (such as secondary currents) there is no reason to exclude the possibility of this influence. For the same reason it may be possible that the depth of flow, y_n, influences ψ (though this seems unlikely as y_n is directly related to the Reynolds number, which is used to compute ψ).

Fig. 1 shows ψ (calculated with slide-rule accuracy) as a function of y_n for different values of S. From the points for runs 84-92 and 93-101 it may be concluded that for these cases ψ is independent of y_n; the other series of points show a trend for ψ to decrease with increasing depth, but the points generally scatter too much to take this for granted. In the writer's opinion the points definitely show that ψ increases with increasing slope; this is the same result as Kirschmer did find.

Regarding smooth tranquil flow the authors found the indeed astonishing result that ψ = 0.93. For the runs concerned (1-16; 102-108) ψ was again

a. Proc. Paper 2018, May, 1959, by Ralph W. Powell and Chesley J. Posey.
1. Civ. Engr., River Research Section, Directorate Upper Rivers, Rijkswaterstaat, The Netherlands.

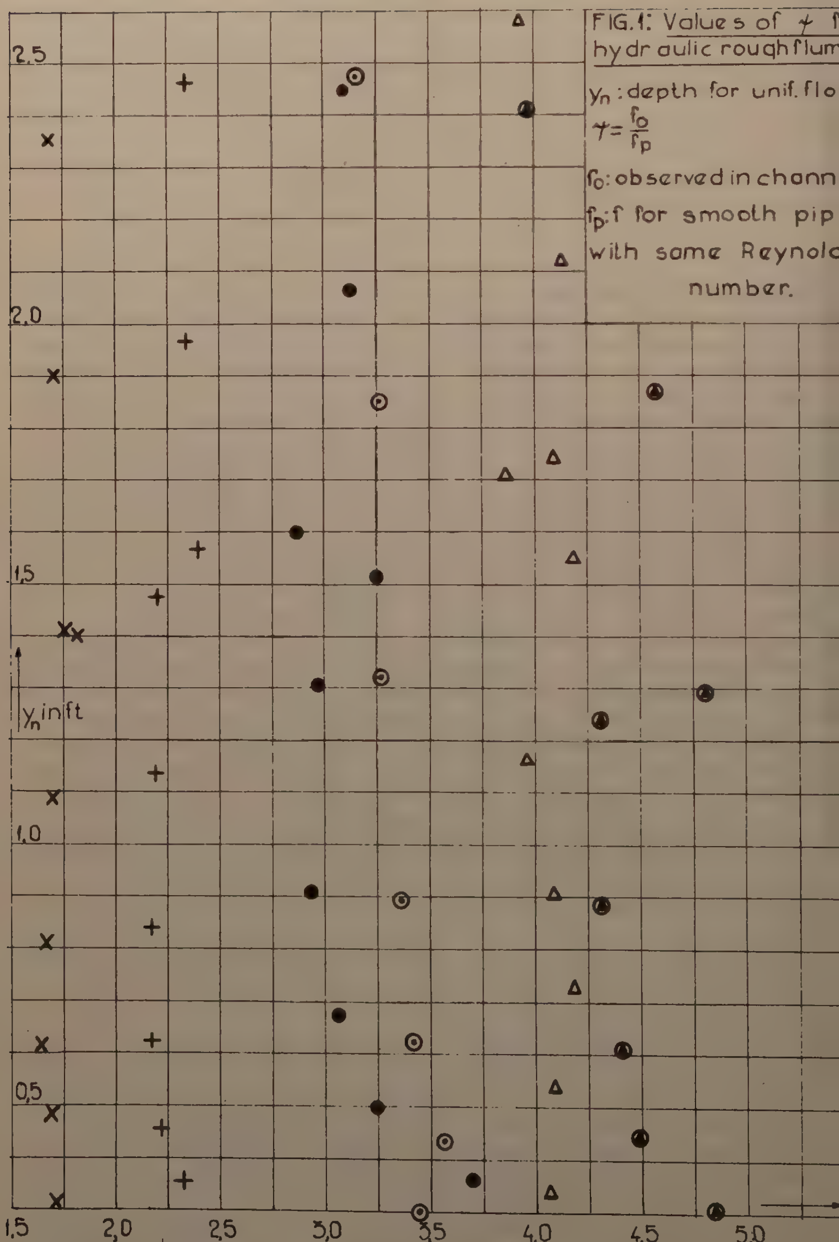


FIG.1: Values of ψ f
hydraulic rough flum
y_n: depth for unif. flo
$\psi = \frac{f_o}{f_p}$
f_o: observed in chann
f_p: f for smooth pip
with same Reynold
number.

Indication	⦿	Δ	⊙	●	+	X
batten spac.	3"	3"	6"	6"	12"	24"
runs nrs.	57–64	65–74	49–55	75–83	84–92	93–
slope x 10^4	19,3–19,6	9,8–10,3	19,4–19,7	10,1–10,2	10,2–10,4	10,2–1
ψ_{av}	4,45	4,05	3,34	3,03	2,25	1,69

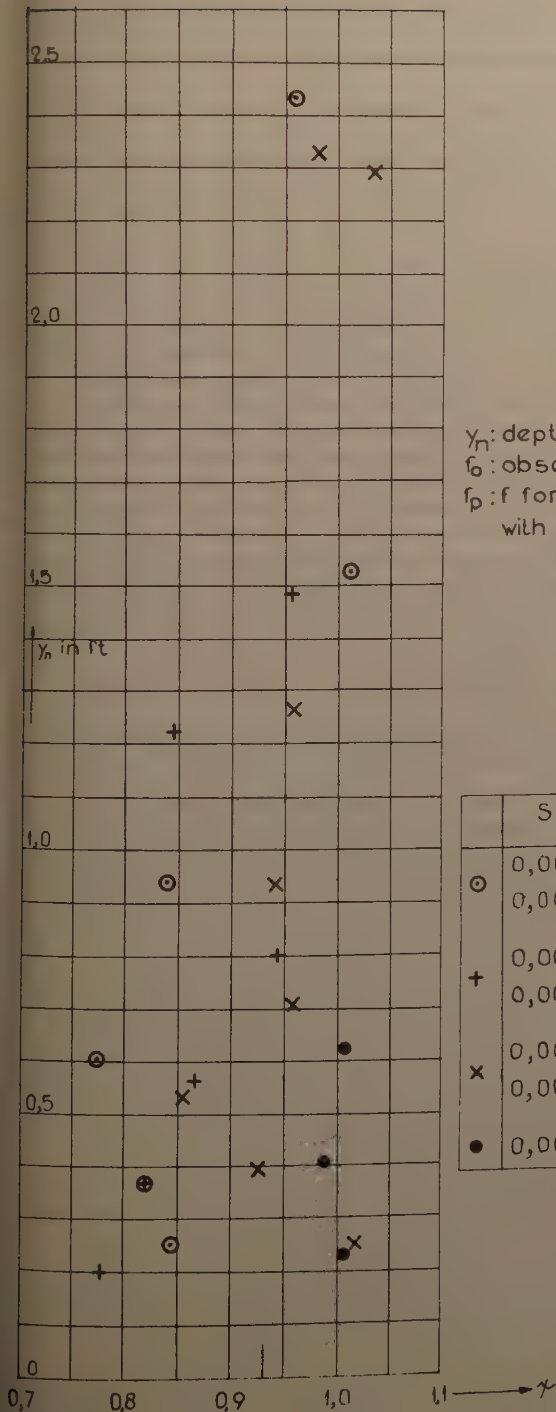


FIG. 2:

$\psi = \frac{f_o}{f_p}$ for hydraulic smooth channel.

y_n: depth for uniform flow;
f_o: observed in channel;
f_p: f for smooth circular pipe with same Reynolds number.

	Slope	run nrs	ψ_{av}	
⊙	0,000448- 0,000480	1-6	0,889	
+	0,000855- 0,000912	7-12	0,868	0,919
×	0,00100- 0,00103	13,102-108	0,957	
●	0,00198	14-16	1,002	

plotted against y_n for different S (Fig. 2). The figure reveals that the low average is mainly determined by low values of y_n, whilst there seems to be tendency for ψ to increase with increasing slope. It doesn't seem justified state anything more definite, but the trend is the same as the one for the hy draulically rough flume.

The values of ψ as found here make it possible to find two equations for if application of Morris' theory[3] is considered suitable: one for S = 8.55 $10.4.10^{-4}$ and one for S = 19.3 to $19.8.10^{-4}$. The relevancy of Morris' form las for practical computations is doubted by the writer as they seem too co plicated; an empirical relation of ψ will suffice for "isolated-roughness flo and "skimming flow".

The results of this analysis indicate that the conclusion of the authors r garding the equal relevancy of the formulas of Prandtl, Manning and Morri for a rough triangular channel may be premature. The writer had no oppo tunity to deal with this question in detail; maybe a definite answer can only given after a new series of tests will have been carried out as even the excellent data given by the authors show for some runs (57-64) a considerabl scattering of points. This question of equal relevancy of formulas may see to be a matter of pure theoretics without much practical meaning for the fl studied, as the mean discrepancies in the mean velocity of all three formu only amounts to 2 to 2,3%, whilst, according to the authors, the probable pe centage error in determining the discharge for some of the runs may amou to the same percentage. When studying river problems however the influe of S on ψ may be of importance and for this reason attention is drawn to so conclusions drawn from further analysis of the data given by the authors.

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3. H. N. Morris: A New Concept of Flow in Rough Conduits. Proc. ASCE, Vol. 80, nr. 390, January 1954.

RESISTANCE PROPERTIES OF SEDIMENT-LADEN STREAMS[a]

Discussions by Tsung-Lien Chou and D. B. Simons and E. V. Richardson

TSUNG-LIEN CHOU,[1] F. ASCE.—The authors are to be highly compliment-ed for taking an important but long neglected problem into the laboratory and the library with special technical dexterity and mathematical insight of eliminating the interwoven elements and of bringing the required factors in focus. The results are neatly tabulated and the conclusions drawn are concise and precise. Since the problem is not only important for practical hydraulic engineers in dealing with river and canal works, but also is closely related with the more theoretical and broader side of boundary layer and turbulence in general, the writer would like to bring out some points which may help clarify the issue.

On Table 4, it is shown that in each set of observation, all important hydraulic characteristics are identical except the slope s and the sediment discharge concentration $\bar{c}$. With these two, the authors attribute a direct cause - effect correlation of friction factor f_b and $\bar{c}$. As repeated on Table 5, the figures are consistent and impressive, though not quite proportional. What this amounts to is that the structure of this part of the problem depends on the accuracy and reliability of the measurement of s which is always delicate and complicated. At this junction, it may be mentioned that in empirical formulas like Darcy-Weisbach $H_f = f\frac{L}{4r}\frac{U}{2g}$ or Chezy $U = C\sqrt{rs}$, the coefficients for c is used to fit in the observed values and they are supposed to take care of all losses which will, in modern hydrodynamical terminology, include losses clue to skin friction, form resistance and turbulence in the pure fluid side and losses in knocking out solid particles from embedment, bringing them into suspension, and accelerating them to the same velocity as the dispersion medium, and host of other losses which are yet unknown. As widely observed, the velocity u varies with space and time, even in a uniform conduit under steady flow. With the empirical relation of u and s as indicated above, synchranically together with u varies the slope s, which may include many ups and downs, even negative values in backflow pockets. In natural streams, only mean values are taken for both. Therefore, in real sense, both u and s are statistical in nature and nothing more than mathematical fictions. Similar are the friction factors, Darcy-Weisbach for Manning n. Furthermore, these factors are not universal constants as shown in the following tabulation. Thus, each friction factor has its own structure and the relations among them are not homogeneous. The fact that they can be used to calculate flow in their special forms does not necessarily mean they can represent certain factors of

a. Proc. Paper 2020, May, 1959, by Vito A. Vanoni and George N. Nomicos.
1. Hydr. Engr., Clinton Bogert Engineers, New York, N. Y.

Empirical Formulas	Chezy	Darcy-Weisbach	Hazen-Williams	Manning
Original Forms	$v = c\sqrt{rs}$	$H_f = f\frac{L}{4r}\frac{v^2}{2g}$	$v = 1.318 c_1 r^{0.63} s^{0.54}$	$v = \frac{1.486}{n} r^{2/3} s$
Friction Factors	c	f	c_1	n
Energy Slope	$s = \frac{v^2}{c^2 r}$	$s = \frac{v^2}{r}\left(\frac{f}{8g}\right)$	$s = \frac{v^{1.85}}{1.667 c_1^{1.85} r^{1.167}}$	$s = \frac{v^2 n^2}{2.203 r^{4/3}}$
For equal s, Ratios of Factors	$c = \sqrt{\frac{8g}{f}}$ $= 1.11 c_1^{0.925}\left(\frac{r^{0.083}}{v^{0.075}}\right)$ $= \frac{1.486}{n} r^{1/6}$	$f = \frac{8g}{c}$ $= \frac{15[illegible].5}{c_1^{1.85} r^{0.165} v^{0.15}}$ $= \frac{3.63 n^2}{r^{1/3}}$	$c_1 = \frac{c^{1.08} v^{0.081}}{1.117 r^{0.093}}$ $= \frac{15.25}{f^{0.54} r^{0.089} v^{0.081}}$ $= \frac{1.372 r^{0.091} v^{0.031}}{n^{1.08}}$	$n = \frac{1.486}{c} r^{1/6}$ $= 1.486 r^{1/6}\left(\frac{f}{8g}\right)$ $= \frac{1.151 r^{0.084}}{c^{0.925} v^{0.075}}$

modern hydrodynamics. Now on Table 4, f is held to count for the variati minute vortices and others. This is apparently outside the function assign to it in the original design.

In applying the empirical formula in a flume like this, s should cover o that portion with established normal flow, and with the entrance and tailwa regions deducted and the cross-section of observation would be set at the geometrical center of the portion. Any average value from tilting of flum bed or the differences of water levels at the ends of the flume may bring i drop-down or back water effect, which would not represent the true value

By solidifying the flume bed after running with sediment, the authors tr to eliminate all factors due to bed configuration in order to emphasize the influence of sediment on hydraulic resistance. It may be worthwhile to re that the turbulence produced on a movable bed is not the same as on simil bed which is fixed, even though the configurations are the same. On a mo bed, turbulence is evened out by picking up solid particles at points of str intensity and dumping some of its load at points of weak intensity, or the distribution of turbulence contours is more or less conformal to the bed t pography. Contrarily, on fixed bed, irregular configuration tends to ampl the turbulence at points of strong intensity and therefore greater turbulen expected on fixed bed. Thus some part of the calculated value might be o by this cause.

The velocity profile shown on Fig. 2 are apparently not taken from the periments of this flume (10-1/2 inch wide). As the depth, the slope and th width are identical for the two profiles, the one with higher velocities wil higher mean velocity and thereby bigger discharge. In that case, more po is required by the flow(7) and there is hardly any strick comparison of th two.

The authors maintain that the suspended sediment reduces hydraulic r ness by its damping effect on turbulence by the following reasoning: "To keep sediment in suspension, i.e. to prevent it from settling due to gravi- tational force, work must come from the vertical component of turbulence

fluctuations and must result in damping of turbulent motion" and "this means that the momentum transfer coefficient is also decreased thus allowing the velocity and velocity gradient to increase."

It should be remembered that turbulence is created at the expense of energy stock of the flow. That is to say, as soon as the turbulence is generated, a certain amount of energy is lost from the flowing stream, no matter whether energy is used in keeping sediment in suspension or in supplying Vortex motion of minute masses and finally dissipated into heat. To keep sediment continuously in suspension, an uninterrupted supply of the necessary energy through turbulence is required. However this damping action may have the effect of preventing the eddies spread out into the entire stream by localizing them to the lower strata. Finally in a stream where the saturated suspension tends to dump its load, there is a chance of releasing some energy for accelerating the flow. These conditions are limited to certain part or certain period of flood where suspended load is dumped.

Furthermore, the transportation processes of solid particles are different for different effective sizes and distributions. Only fine particles less than 0.2 mm can be thrown up into suspension by and move with turbulence generated purely by hydraulic resistance. On the other hand, bigger particles create extra turbulence by their projections into flowing stream and produce suction effect helping them into sliding, rolling and saltation. This demands more energy consumption and thereby increases the hydraulic resistance. The above conditions are well observed in pumping sands in suspension and suspension tests.

In addition to the direct experimentation in laboratory, the authors collect some interesting information of field data from national streams. Here the direct cause-effect correlation of f and $\bar{c}$ is further overshadowed by additional factors of (a) source of solid supply, varying with the topographical and geological conditions of the river valley, (b) hydraulic properties of the stream, aggrading or degrading. The mutual action and reaction of these factors can easily produce contradictory effects.

In general, a rising stage has higher average velocities for two reasons. First, in rising stage, the energy slope is a drop-down curve with increasing velocities towards each downstream section. Thus the velocity is accelerated. Secondly, the potential head is converted into kinetic head with smaller loss.

In most cases, the amount and the intensity of sediment load vary with the stage. If a freshet is originated from an upstream watershed and gains strength in flowing downstream, it will pick up silt to its optimum capacity. This happens very often in the rising stage. After passing over the peak flow, the river tends to dump its suspended load. However, there may be exceptions to this rule in dealing with difference sources of solid supply. For instance, on the Yellow River at Tung-Kuang, the intensity of suspended sediment varies with the source of flood. If it comes from the north bank tributaries (Shansi Province) where steeper grades of river beds with thicker eolian deposits of loess, the River has a tendency to dump its load even in rising period. The contrary is true, if the flood is started from the south side. Similar situation is observed at Chingling Chi on the Yangtze River where the discharge from the south is clarified by passing the Tung-Ting Lake which acts as settling basin.

The above few examples just illustrate the complicated relations of various factors in natural streams and the cause-effect correlations can easily be reversed, unless comprehensive assessments are made for some period.

On the theoretical side, the sediment can reduce hydraulic resistance i the following ways:

(1) Smoothing up the surface roughness by scouring off the mound and f up the depressing
(2) Evening off the aquatic growths
(3) Exposing fresh surfaces which are more pliable
(4) Compacting particles by selective deposition and distribution
(5) Keeping down the vertical component of turbulence (as the authors' main item). Again, every factor must be properly appraised.

The problem of suspended load hydraulic resistance is complicated but very interesting. The authors have opened up a new vista for both practic and theoretical investigation. Perhaps, another way of attack may be the rect measurements of the turbulence itself for the similar condition of tu and clear flows.

D. B. SIMONS,[1] M. ASCE and E. V. RICHARDSON,[2] A. M. ASCE.—The authors are to be commended for their excellent explanation of many of th important aspects of the mechanics of flow in alluvial channels. Based up similar studies conducted in both large and small flumes by the writers, t equipment used and a few of the concepts presented by the authors are ela rated on.

One of the problems associated with a study of this type is the difficult evaluating the effect of flume size on data and observed flow phenomenon. The writers found that the results obtained with a given bed material usin large recirculating flume 150 feet long, 8 feet wide, and 2 feet deep, could be reproduced in a smaller recirculating flume 60 feet long, 2 feet wide a 3 feet deep. For example, the forms of bed roughness observed in the or of increasing shear in the small flume were:

1. Plane bed before beginning of motion
2. Plane bed after beginning of motion
3. Dunes
4. Transition from dunes to plane bed and standing waves
5. Standing sinusoidal sand and water waves which were in phase
6. Antidunes

Using the same bed material in the large flume the forms of bed rough in order of increasing shear were:

1. Plane bed before beginning of motion
2. Ripples
3. Ripples superposed on dunes
4. Dune
5. Transition from dunes to plane bed and standing waves
6. Standing sinusoidal sand and water waves which were in phase
7. Antidunes.

The large flume produced the forms of bed roughness observed in the whereas, the small flume did not. It was also noted that the height of dun

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related to the depth of sand bed. As depth of bed material is decreased the amplitude of the dunes is decreased. These differences in results, over and above those which can be compensated for by correcting for side wall effect, present a serious problem to those who are trying to utilize data collected by the various investigators, particularly that data gathered in very small flumes.

No mention is made by the authors of the fact that the form of bed roughness is related to size and gradation of bed material. This may be due to the limited range of conditions which they investigated. To illustrate using sand sizes ranging from d = 0.2 mm to d = 0.5 mm the writers have observed that the amplitude of the dunes is independent of size of bed material but the spacing and shape of the dunes are related to size of bed material. The spacing of the dunes increases with decreasing size and the angle the fore plane of the dune makes with the horizontal decreases with decreasing size of bed material. The net result is that resistance to flow is smaller with the fine sand than with the coarser sand by as much as 20 per cent. This discussion perhaps raises the question of bed form terminology. To avoid confusion, the major forms of bed roughness which have been observed in alluvial channels by the writers are illustrated in Fig. 1.* From this figure, it is apparent that the form of bed roughness described by the authors, according to this classification, is ripples.

Referring to the difference in the bed friction factor for sediment-laden flow over a loose sand bed and clear water flow over a fixed sand bed, of the same configuration, it is doubtful that the total difference can be explained by

*From "Forms of Bed Roughness in Alluvial Channels" by D. B. Simons and E. V. Richardson, personal communication.

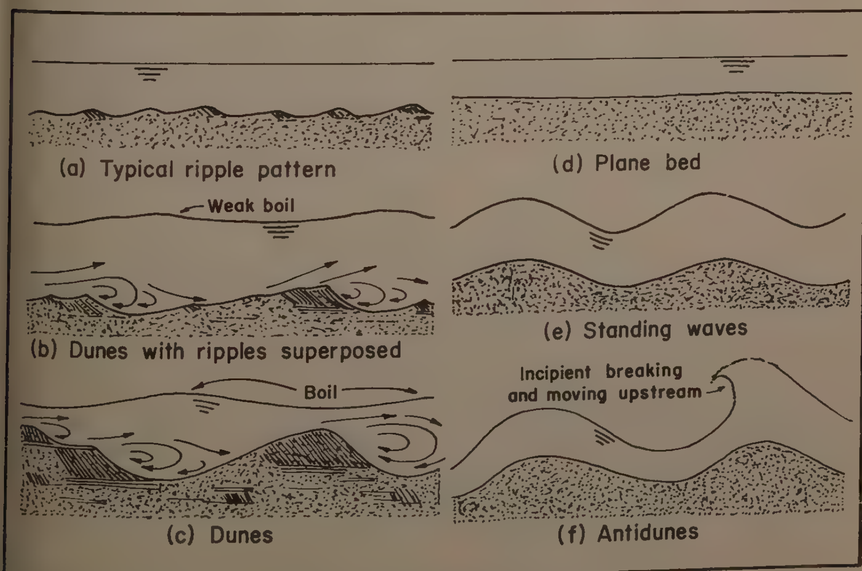

Fig. 1 Forms of Bed Roughness Observed in Alluvial Channels

the presence of the suspended sediment. The writers have observed that t scale and intensity of turbulence are entirely different on a rigid bed with bed material transport than on a loose sand bed having the order of 10 ppn of bed material transport, none of it in suspension, and other flow factors same.

The effect of sediment load on resistance to flow is significant as illust ed. However, the authors conclusion that all of the computed decrease in sistance to flow can be attributed to the damping effect of the suspended se ment on the turbulence is not concurred in. The presence of fine sediment causes effects on fluid properties which change the entire fluid bed materi interrelationship. These factors appear to be of more importance than the dampening effect of the suspended sediment.

The presence of very fine sediment (clays) in the water also influences sistance to flow in alluvial channels. Based upon recent studies by the wri concentrations of fine sediment on the order of 40,000 ppm can reduce re-sistance to flow as much as 40 per cent in the tranquil flow regime. Howe under rapid flow conditions, the presence of fine sediment can increase re sistance to flow.

The validity of the statement on page 104 which explains the existence c smaller friction factor for the same discharge on a rising stage than on th falling stage, where the decrease in friction factor is presumed to result f a readjustment of the bed due to the larger sediment load is questioned. I not more probable that the increase in load and the reduction in friction fa are both related to the fact that with the increase in shear with stage the b form changes and that the change in bed form will lag the change on stage

To better cope with the problems associated with fully developing and u izing our water resources, additional work of this caliber, preferably done larger flumes, should be initiated. Only by making studies of this type can hope to explain the quantitative effect of size of bed material, gradation of material, temperature, geometry of channel, depth of sand bed, fine sedim load, and rate of change discharge with time on total bed material transpo the forms of bed roughness which develop, and the related resistance to fl

HYDRAULIC MODELS OF THE ST. LAWRENCE POWER PROJECT[a]

Discussions by G. T. Berry and G. B. Fenwick and F. R. Brown

GEORGE T. BERRY,[1] A. M. ASCE.—The St. Lawrence Power Project was unusual and challenging in many respects. Although the St. Lawrence River normally reaches its minimum annual discharge during January or February, the average January discharge for 98 years of record is 221,000 cfs and the average February discharge is 218,000 cfs. Comparing these averages with the average annual discharge of 246,000 cfs it is apparent that a low flow season for diversion or closure does not really exist. To divert such large flows required that the construction be performed in several stages.(1) At each construction stage the depth of water in the navigation channels and canals in the project area had to be maintained. The water surface elevation of Lake Ontario could not be altered from its natural condition and the discharge to downstream power plants could not be impaired.

The hydraulic model studies described by Mr. Bryce focused attention on the salient features of possible alternate solutions and permitted the hydraulic engineers to experiment with these variations until the optimum design was achieved.

This series of model studies is an outstanding example of the technique of complementing the general results of a small scale model by investigation of the critical details in larger scale models. The best location for the structures and the location, size and shape of the channels required by navigation and ice cover considerations were determined in the river models. These were at the scale of 1:500 horizontally and 1:100 vertically which is quite large for models of such a large river. Details of the stilling basins and gates were then studied in 1:50 scale partial models. Finally comprehensive models of Long Sault and Iroquois with enough upstream and downstream river channel to simulate prototype approach and discharge conditions were built and tested to determine the flow pattern and hydraulic performance for each construction stage.

Designs of the energy dissipators at Long Sault and Iroquois were developed from a long series of tests for each. Several forms of baffles and sills were tried. The final form which had no baffles and a solid end sill was the results of careful experimentation to obtain the lowest possible velocity in the channels below the dams. The ineffectiveness of baffles in these spillways which have high entrance velocities agrees with experiments conducted a few years ago at M.I.T. on the effects of Baffle Piers by Harleman.(2) The excellent performance of the Long Sault stilling basin can be judged from the fact that, although the natural tailwater depth was insufficient to form a

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a. Proc. Paper 2022, May, 1959, by John B. Bryce.
1. Civ. Engr., Chas. T. Main, Inc., Boston, Mass.

hydraulic jump, a jump was forced to form within the basin for the three different diversion conditions of (a) diversion through low sluices, (b) di- versions through ports left in the dam and (c) flow over the completed da The channel downstream from the dam was rock in which the natural pre project velocity was in the same range as those achieved with the recom ed spillway design. Similarly satisfactory energy dissipation was achiev Iroquois Dam.

The remarkable reduction of the hydraulic downpull effected by resha the bottom of the Long Sault port gates illustrates how important this fac is in designing gates and their hoisting equipment. Unbalanced hydraulic es on gates of this type depend primarily on the local conversion of pres head to velocity head in the restriction beneath the gate. The shape of th bottom of the gate and the approach conditions determine the magnitude pressure reduction and the area over which it acts. Size of gate, depth c mergence, and location of skin plate and seals determine the magnitude hydrostatic force which is not counterbalanced by vertical force on the b of the gate. The tests described cover a variety of gate conditions with stream and downstream skin plates and with various depths of submerge It is believed that structural and hydraulic engineers would find the data valuable if the author would show the physical details and dimensions of gates, the pertinent boundary conditions and the head on the gate along w uplift and/or downpull at various gate openings. Available data correlat these factors are scarce.

It is impossible to accurately isolate the savings in construction cost tributable directly to the model studies. The substitution of dry excavat south of Gallop Island for wet excavation along the north channel resulte saving of about $1.00 per cubic yard on almost 6 million cubic yards of e vation. This alone repair manifold the cost of the model studies. Of equ importance was the fact that by these accurate model tests, the design v ties for both navigation and ice packing criteria could be met with confid so that very little margin for unknown or unexpected conditions was neec The definite detailed construction procedure which was developed and ou in the specifications and the model demonstrations, which were in some made for the bidders to show the construction sequence and hydraulic co ditions which would be encountered, undoubtedly resulted in substantial in cost.

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G. B. FENWICK,[1] F. ASCE and F. R. BROWN,[2] F. ASCE.—Mr. Bryce's paper portrays a classic example of the utilization of hydraulic models as a fundamental part of the design and construction of a major engineering project involving a wide variety of complex interrelated features. Wide experience has established the fact that model studies are an indispensable adjunct to analytical procedures in the development of optimum designs and construction sequences for major hydraulic channels and structures involving complex features and large expenditures of funds, and that model studies almost invariably produce major design improvements at costs that are disproportionately low in relation to project costs. For many hydraulic engineering problems, including some of the features of the St. Lawrence project, hydraulic model studies constitute the only reliable approach to satisfactory solutions.

It is noted from Mr. Bryce's paper that his lowermost river model (Model No. 9), utilized for studies in the powerhouse tailrace, terminated just below the head of Cornwall Island and included only the upper ends of the channels north and south of that island. He mentions that a portion of the excavation of the south channel, which contains the navigation channel approaching the Eisenhower Lock was accomplished by the United States Seaway Development Corporation. It might be of interest to note that two river model studies were conducted by the U. S. Army Engineer Waterways Experiment Station to develop optimum excavation and spoil-placement plans and construction sequences for the navigation channel in the river from Barnhart Island downstream to Lake St. Francis.

An interesting scale-distortion problem in connection with the modeling of this reach arose from the fact that 60 per cent of the river flow turns sharply to the south around the head of Cornwall Island and enters the navigation channel below the lock almost normal to the east-west navigation course. It was therefore necessary, among other things, to design a curved deflecting dike to realign this southward flow with the east-west navigation channel to eliminate intolerable cross currents in the latter. One model of the overall reach, with linear scales of 1:300 horizontally and 1:100 vertically, was used to develop plans throughout the Barnhart Island - Lake St. Francis reach, except in the vicinity of the above-mentioned sharp changes in flow direction near the head of Cornwall Island. Since it was considered that model scale distortion would introduce significant errors in the latter area owing to the sharply changing flow directions, this immediate area was duplicated in another model with an undistorted linear scale of 1:100. As was expected, comparisons of detailed data from both models demonstrated that the distorted model produced erroneous distributions of velocities in the vicinity of the sharp direction changes while data in other areas checked well in both models. Accordingly, the two models were used together very effectively to develop plan details for the overall reach.

The problem of energy dissipation downstream from the St. Lawrence structures is unique in that large discharges must be passed during the construction stage. On many other rivers, seasonal fluctuations are such that critical stages of construction can be accomplished when river flows are low. Because of insufficient tailwater it was necessary to force a hydraulic jump

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for diversion flows and for final operating conditions. In the case of the Sault Dam, this appears to have been accomplished by use of a 12.5-ft-hi end sill. However, a question arises as to the necessity for a 40-ft-wide A lesser width sill and longer apron at elevation 140 might improve cond Provision of a steeper slope at the toe of the overflow section or sloping diversion ports are other alternatives for improvement of energy dissipa

The energy dissipator for the Iroquois Dam appears to be designed on diversion flows. It was stated that only ten sluices (or gate bays) were p tected, which raises the question as to the treatment of the bed of the str immediately downstream from the other 22 sluices. If economies were a ed by the omission of any protection below 22 sluices, the author is to be mended. Many times engineers are reluctant to omit any part of the ene dissipator, even where conditions are such that it can be done safely.

The effect of gate operation on energy dissipation is mentioned only b ly by Mr. Bryce. However, various combinations of gates and openings c have an important bearing on the strength and magnitude of river current and on the design of an energy dissipator. On many river dams, gate ope ation to secure satisfactory energy dissipation is one of the most import phases of a model study.

Another important phase of the St. Lawrence work reported upon by M Bryce concerns the hydraulic loading of gates. This topic could be the s of an entire paper. The Waterways Experiment Station has made observ similar to those indicated in Fig. 16, both in the model and in the field.(1 However, in order to avoid too much uplift, a 45-degree lip is usually us Efforts to use a gate with upstream seals and the upstream skin plate pr ing below the bottom members were successful as far as hydraulic loadi the gate was concerned. However, the expansion of the jet laterally into gate slot intensifies the possibility of cavitation of the conduit liner.

For submerged gates with downstream seals, the depth of water over gate contributes materially to the downpull force. As the gate is raised the top seal broken, the clearance on the downstream side of the gate be significant. The relative clearances on the upstream and downstream si can maintain or release the column of water over the gate, and thus chan the magnitude of the loading. It has been found best to maintain some wa in the gate well over the gate for more positive control of the gate.

No mention was made of studies of the vibration characteristics of th Lawrence gates. With a flat lip, a gate tends to vibrate vertically in res with the vortex trail shed from the upstream edge. With a 45 or 60 degr the vortices are shed from the downstream edge of the gate, thus elimin any areas on which pressure pulsations can act. The shape of the lips o gates in Fig. 17 cannot be determined, although on the basis of the tests diversion port gates at Long Sault Dam, it is assumed that they are eith or 60 degrees. Other contributing factors to the hydraulic loading of ga are venting downstream from the gate and entrapment of water in the ga members.

Another problem area not mentioned in Mr. Bryce's paper concerns t emergency closure of ports or gated areas by means of stop logs or a se of built up members. These devices are designed to be lowered through ing water to provide a barrier to flow in the event of damage to a gate. tests and field experiences indicate that it is almost impossible to insta emergency closure devices in flowing water. Flow over the top of and b neath the stop logs or built up steel members sets up vortex trails which

the closure devices to oscillate violently in a vertical direction. The amplitude of movement is such that the closure device could be damaged or wedged in a skewed position.

It is realized that space requirements prevented the author of the basic paper from going into detail on specific problems. However, it is hoped that in his closing remarks the author can expand on his tests to develop energy dissipators and investigate the hydraulic loading of gates.

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GROUND-WATER PROBLEMS IN NEW YORK AND NEW ENGLAND[a]

Discussion by C. Biemond

C. BIEMOND.[1]—As stated in the introduction to the paper, there is generally interchange between surface water and ground water under natural conditions. There can also be interchange by artificial processes: surface water can be brought to intermingle with and to diffuse in ground water by artificial replenishment, as practised in Sweden and in The Netherlands. The ground water captured in works of this nature is a mixture of natural ground water and artificial ground water, which has in many respects the same quality as natural ground water. Artificial recharge is accomplished in these cases by spreading in shallow ponds or canals. The writer does not know of examples in Europe of recharge with surface water by means of wells.

Intermingling of ground water and surface water is also brought about by what is called "Uferfiltration" in Germany and practised on a large scale along the Ruhr River and the Rhine River. From the paper it is deduced that, in the case of the Blackstone River near Woonsocket, R. I., this process, in which wells for the withdrawal of ground water are placed near or in the river bed, is being envisaged.

It is the purpose of the writer to report on some features resulting from this technique, that might be of interest.

The alluvial deposits in and along river beds—where the river has been aggrading—consist as a rule to considerable depths of a mixture of boulders, gravel and coarse sand—a favorable condition for ground-water withdrawal. When the wells are placed near the river and reach into the underlying deposits, the water withdrawn would partly percolate out of the natural ground-water body adjacent to the river bed, but, as a result of the hydraulic flow pattern, mostly through the river bed, and therefore would consist for the most part of ground water that was recently surface water in an open river. This water would contain the natural impurities of river water, such as silt and decayed algae, but also the products of artificial pollution, which might be organic substances, ammonia, iron, and others. Furthermore, there may be, as a result of heavy pollution, a very low oxygen content.

Pollution is kept away from the Ruhr River as much as possible; water percolating through the gravel and sand in the river bed reaches galleries and wells in a satisfactory condition.

The Rhine River is in a state of increasing pollution and the water withdrawn from wells bordering the river bed is gradually giving more trouble. The troubles consist firstly in deteriorating quality of the extracted water and secondly in a gradual clogging of the river bed. Deterioration of quality is

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a. Proc. Paper 2056, June, 1959, by Joseph E. Upson.

1. Director Amsterdam Waterworks, The Netherlands.

particularly apparent in the taste. Clogging, in the opinion of the writer, be caused by high entrance-velocities for the water penetrating into the al luvial deposits, thus introducing insoluble particles into the interstices of deposits. It might also be caused by introducing oxygen-poor water into a medium which doubtless contains ferric deposits that might thus go into s lution, to be deposited subsequently in interstices where intermingling wit oxygen-rich water would take place.

In the opinion of the writer artificial replenishment, in comparison wit wells located in alluvium along rivers, has certain advantages that may b worthwhile to consider.

FLOODS OF THE FLORIDA EVERGLADES[a]

Discussion by J. R. Bowman

J. R. BOWMAN,[1] M. ASCE.—There are so few large inland bodies of water in the United States that are subjected to hurricane winds that opportunities to compare their tidal characteristics are rare. Pamlico Sound, in eastern North Carolina, is one of the few such waters that can be likened to Lake Okeechobee. This sound is land-locked by virtue of a string of sand barrier reefs, known as the Outer Banks, which separate the sound from the Atlantic Ocean. It has a surface area of nearly 2,000 square miles, and it drains an area of some 25,000 square miles in Virginia and North Carolina. Its principal outlets to the ocean are three relatively narrow and shallow inlets through the Outer Banks; thus the normal sound levels can be considered to be independent of ocean tides, except for local effects in the immediate vicinities of the inlets. Unlike Lake Okeechobee, Pamlico Sound is not a "huge saucer", but assumes the form of a rough parallelogram bounded on the north and west by the relatively steep shores of the marshy flatwoods on the mainland, and on the east and south by the shoal waters contiguous to the Outer Banks. The area of maximum depth, varying from 20 to 25 feet, is found in a narrow reach along the northeast-to-southwest major axis of the sound.

Hurricanes are not strangers to eastern North Carolina. In fact, a list of these storms that have directly affected Pamlico Sound alone since 1900 would be as long as the author's list associated with his Fig. 6. Analysis of Fig. 6 and its companion list shows that the Okeechobee area is crossed by two general groups of hurricane paths. One is associated with storms from the Atlantic Ocean that generally strike southeastern Florida prior to their recurvature. Storms of the other group approach from the Caribbean Sea and the Gulf of Mexico, striking the west side of the peninsula after having recurved eastward. It is noted that the highest wind speeds generally have been more closely associated with the storms of the former group. Because of the orientation of the Florida peninsula in relation to its watery environs, excessive rainfall is not necessarily peculiar to either group. However, it is interesting to note that the 1924 hurricane produced exceptionally heavy rainfall in areas north of Lake Okeechobee, whereas the storm center passed at least 50 miles south of the lake; the pronounced eastward swing in the path of this storm (number VI) suggests the proximity of a stationary front, which could account for the widespread area of excessive precipitation.

Because Pamlico Sound is situated some eight degrees north and five degrees east of Lake Okeechobee, the majority of its hurricanes approach from the Atlantic Ocean during or after recurvature. When a hurricane recurves

a. Proc. Paper 2058, June, 1959, by E. W. Eden, Jr.

1. Civ. Engr., Erik Floor & Assocs., Inc., Chicago, Ill.

in the higher latitudes, its forward speed usually increases considerably, carrying the storm rapidly past or across the sound; therefore, the regions maximum winds do not have sufficient time in which to develop severe wind tides over the longer fetches, unless the radius to the maximum wind field i exceptionally large. Three of the most severe storms in this region have amply met this requirement; along the Outer Banks on the south side of Pamlico Sound, the winds and sound tides of the "Great Atlantic Hurricane" of 1944 are remembered as the most severe in half a century.

As a tropical storm enters its zone of recurvature, it is not unusual for forward speed to decrease somewhat; on occasion, a blocking ridge of high pressure will cause the storm to stall and wander aimlessly. Hurricane Io of 1955 reached a virtual standstill immediately after crossing the North Carolina coastline in the vicinity of Cape Lookout (located about 30 miles d south of the western end of the sound); it then proceeded to drift slowly and erratically westward, covering a path distance of only 50 miles in nine hour before veering northeastward and picking up speed. During the "stagnant" period, the western portion of Pamlico Sound experienced winds of 75 to 10 miles per hour, and storm tides reached unprecedented heights in many mu cipalities in that area.

Not infrequently the varying conditions of surface friction presented to a hurricane passing over land masses will cause the eye of the storm to beco distorted near the surface, thereby altering surface wind patterns. The sca city of surface observations made during the passage of Hurricane Barbara 1953 leaves the writer somewhat uncertain as to the configuration of the surface eye immediately after it crossed the coastline in the immediate vic ty of Cape Lookout. The storm was an infant, as hurricanes go, its maximu steady winds just reaching hurricane force. Weather Bureau summaries of the storm do not suggest the presence of unusual characteristics, nor does reconstructed path, based on available information, reveal significant irreg larities. Had the eye passed undistorted across the western part of Pamlic Sound, the maximum setup should have occurred along the north shore. Al though flooding did occur where expected, the highest setup appeared to hav occurred along the northern portion of the east shore of the sound; the writ observed a fresh high water line, estimated at 10 to 12 feet above normal s level, extending for several miles. Moreover, structural damage, which wa largely confined to an area on the Outer Banks about 20 miles further north all occurred within the same half-hour period; the damage was inflicted by southeast wind at least 60 miles in advance of the apparent eye. Curiously enough, the path of the storm eventually passed 25 miles west of the damag zone, but the eye was not observed within 10 miles of that area.

Florida also has experienced some unusual behavior on the part of hurri canes; among the more recent examples were the small but intense Miami storm and the looping Cedar Key storm, both in 1950.(1) The former (numb XVIII in Fig. 6) maintained a tight circulation pattern around a very small from Miami to Lake Okeechobee; however, the published observations indic a considerable reduction in wind speed in the Okeechobee area, with winds only gale force observed in the vicinity of the lake. After crossing the lake the eye broadened and full hurricane intensity was regained.

Although the Cedar Key storm of September, 1950, was well removed fro the Everglades, the writer wonders how it might have affected the project area, assuming the loop, the subsequent path and the associated precipitatio pattern were superimposed on Lake Okeechobee and the Everglades. Cedar

Key received 25.2 inches of rain in a little over 24 hours, and depths of 7 to 20 inches were recorded elsewhere along the path of the storm.

The author's estimates as to the effectiveness of the Central and Southern Florida Flood Control Project are impressive, but one wonders whether the general public fully appreciates their value. In the writer's contact with municipal flooding problems, he has observed that having assumed a considerable obligation in approving a bond issue, one derives small comfort from the knowledge that the new storm relief system probably will reduce the depth of water in his basement from four feet to only two.

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ROLL WAVES AND SLUG FLOWS IN INCLINED OPEN CHANNELS[a]

Discussion by F. F. Escoffier

F. F. ESCOFFIER,[1] M. ASCE.—The author has made an interesting study of certain waves of instability that occur in fairly shallow water. Apparently either surface tension or laminar flow plays an essential part in the generation of these waves. In the interest of providing a somewhat more complete picture of the subject of waves of instability the writer feels that it may be helpful to present his understanding of how some waves of instability originate.

The type of waves the writer has in mind normally occurs in relatively deep and fully turbulent flow. An interesting photograph of such a roll wave having a height of five feet was published in 1936 by W. H. Holmes(1) together with some rather detailed measurements on this and other roll waves observed by him. News is received from time to time of highly destructive roll waves that occur in canyons in the western part of the United States. These are generally referred to as "walls of water."

The writer will use substantially the same reasoning that was used in 1950(2) except that in the interest of simplicity it will be assumed that there exists a uniform distribution of velocity in the channel cross-section, an assumption that greatly simplifies the results.

The dynamic equation and the equation of continuity, for a uniform channel, which serve as a starting point for the derivation are

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} + g\frac{\partial y}{\partial x} = gs\left(1 - \frac{v^2}{v_o^2}\right) \qquad (1)$$

and

$$w\frac{\partial y}{\partial t} + A\frac{\partial v}{\partial x} + vw\frac{\partial y}{\partial x} = 0 \qquad (2)$$

where V is the mean velocity, t is time, x is the distance measured downstream, g is the acceleration of gravity, y is the depth of water at the lowest point of the cross-section, S is the bottom slope, V_o is the normal velocity, w is the water-surface width, and A is the cross-sectional area.

Eq. (2) is now multiplied by r and the result is added to Eq. (1) to obtain

$$\frac{\partial v}{\partial t} + (V + rA)\frac{\partial v}{\partial x} + rw\frac{\partial y}{\partial t} + (g + rvw)\frac{\partial y}{\partial x} = gS\left(1 - \frac{v^2}{v_o^2}\right) \qquad (3)$$

The condition for the left-hand side of Eq. (3) to be a directional derivative in the (x,t) plane is

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a. Proc. Paper 2085, July, 1959, by Paul Mayer.

1. Hydr. Engr., Mobile Corps of Engrs., Mobile, Ala.

$$\frac{1}{v + ra} = \frac{rw}{g + rvw}$$

which yields the following solution for r

$$r = \pm\sqrt{\frac{g}{wa}}$$

Substituting this back into Eq. (3) leads to the equation

$$\frac{\partial v}{\partial t} + \left(v \pm \sqrt{\frac{ga}{w}}\right)\frac{\partial v}{\partial x} \pm \left[\sqrt{\frac{gw}{a}}\frac{\partial y}{\partial t} + \left(\pm\sqrt{\frac{ga}{w}} + v\right)\sqrt{\frac{gw}{a}}\frac{\partial y}{\partial t}\right]$$
$$= gS\left(1 - \frac{v^2}{v_o^2}\right)$$

or

$$\frac{\partial v}{\partial t} + (v \pm c)\frac{\partial v}{\partial x} \pm \left[\frac{\partial w}{\partial t} + (v \pm c)\frac{\partial w}{\partial x}\right] = gS\left(1 - \frac{v^2}{v_o^2}\right)$$

where

$$c = \sqrt{\frac{gA}{w}}$$

and

$$w = \int_0^y c\,\frac{dA}{A}$$

w is a stage variable which is used instead of y to represent the level o the water in the channel. In a rectangular channel it assumes the familiar form

$$w = 2c$$

If

$$z = v \pm w$$

and

$$F = gS\left(1 - \frac{v^2}{v_o^2}\right)$$

are substituted into Eq. (5) there is obtained the equation

$$\frac{\partial z}{\partial t} + (v \pm c)\frac{\partial z}{\partial x} = F \qquad ($$

Use is now made of the identity

$$dz = \frac{\partial z}{\partial t}dt + \frac{\partial z}{\partial x}dx$$

to eliminate the partial derivative $\frac{\partial z}{\partial t}$ from Eq. (10) with the result

$$\frac{dz}{dt} - \frac{\partial z}{\partial x}\left[\frac{dx}{dt} - (v \pm c)\right] = F$$

from which it can be seen that along the path in the (x,t) plane defined by

$$dx = (v \pm c)\, dt \tag{12}$$

the expression in brackets disappears and

$$dz = F\,dt$$

which in the original terminology becomes

$$d(v \pm w) = gS\left(1 - \frac{v^2}{v_0^2}\right) dt \tag{13}$$

Eqs. (12) and (13) are the equations of characteristics. The characteristic corresponding to the upper or plus sign represents a pulse or wave point traveling downstream with a velocity V + c and shall be designated a forward characteristic. The one corresponding to the lower or minus sign represents a pulse or wave point traveling with a velocity V - c, and consequently moving either upstream or downstream accordingly as V is less than or greater than c. It shall be designated a backward characteristic.

It is now assumed that small variations δV, δV_0, and δw occur in an otherwise uniform flow in which

$$V = V_0$$

and, therefore

$$\delta\left[gS\left(1 - \frac{v^2}{v_0^2}\right)\right] = \frac{2gS}{V}(\delta v_0 - \delta v) \tag{14}$$

Along the forward characteristic

$$d(\delta v + \delta w) = \frac{2gs}{V}(\delta v_0 - \delta v)\, dt \tag{15}$$

and along the backward characteristic

$$d(\delta v - \delta w) = \frac{2gS}{V}(\delta v_0 - \delta v)\, dt \tag{16}$$

In Fig. 1 the lines MP and BO represent forward characteristics and the line Op a backward characteristic. The zone to the right of BO is assumed to be undisturbed uniform flow. A pulse or disturbance starts at M and moves downstream along MP. Ordinarily the effect of channel friction will be to reduce the magnitude of this pulse as it moves downstream. However, in unusually steep channels the opposite may be true and the pulse will continue to increase in magnitude. In such a case the flow is said to be unstable.

The line OP has been taken short enough for the right-hand side of Eq. (16) to be a negligible quantity so that

$$\delta v - \delta w = 0 \tag{17}$$

Eq. (17) is now used to eliminate V from Eq. (15) with the result

$$d\delta w = \frac{gS}{V}(\delta V_0 - \delta w)\, dt \tag{18}$$

Since V_0 is a function of w it is possible to write

$$\delta V_0 = \frac{dV_0}{dw}\,\delta w \tag{19}$$

The substitution of Eq. (19) into Eq. (18) yields

$$d\delta w = \frac{gS}{V}\left(\frac{dV_o}{dw} - 1\right) \delta\, wdt$$

An examination of Eq. (20) will show that δw will increase or decrease magnitude as it moves downstream accordingly as $\frac{dV_o}{dw}$ is greater than or than one. The requirement for the formation of waves of instability is, th fore

$$\frac{dV_o}{dw} > 1$$

Eq. (20) can be integrated. The result is

$$\delta w = C\, e^{\frac{gS}{V}\left(\frac{dv_o}{dw} - 1\right) t}$$

where C is a constant of integration.

The same line of reasoning with the roles of the forward and backward characteristics interchanged yields the formula

$$\frac{dv_o}{dw} < -1$$

as the requirement for the formation of waves of instability along the bac characteristics. This is an unusual condition in that it requires the norm

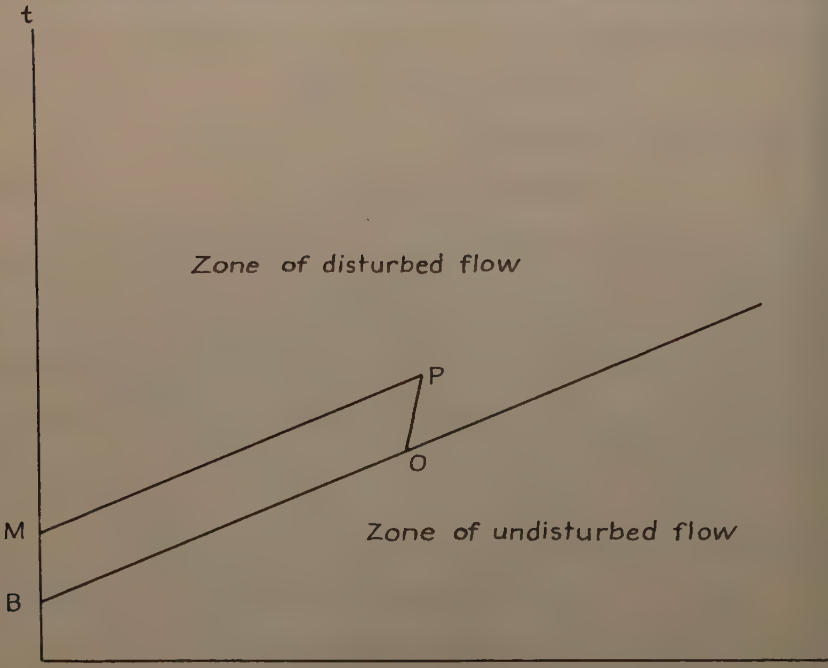

Fig. 1

velocity V_o to diminish as the stage w increases. However, it is a condition that can be met in a closed conduit flowing nearly full.

It remains to show that Eq. (21) will upon appropriate substitution yield the more familiar formulas in general use. In a rectangular channel of infinite width

$$V_o = k \sqrt{S}\ y^n \tag{24}$$

and

$$w = 2 \sqrt{gy} \tag{25}$$

Therefore

$$\frac{dV_o}{dw} = \frac{k \sqrt{S}\, n\, y^{n-1}\, dy}{\sqrt{g}\ y^{-\frac{1}{2}} dy} = \frac{k \sqrt{S}\, n}{\sqrt{g}}\ y^{n - \frac{1}{2}} \tag{26}$$

The formula for the critical slope obtained by equating c as given by Eq. (6) to V_o as given by Eq. (24) is

$$S_c = \frac{g}{k^2}\ y^{1-2n} \tag{27}$$

Eliminating g, k, and y between Eqs. (26) and (27) there is obtained

$$\frac{dV_o}{dw} = n \sqrt{\frac{S}{S_c}} \tag{28}$$

Eq. (21) becomes

$$n \sqrt{\frac{S}{S_c}} > 1$$

whence

$$S > \frac{S_c}{n^2} \tag{29}$$

For the Chezy formula $n = \frac{1}{2}$ and Eq. (29) becomes

$$S > 4\ S_c$$

which is the result obtained by Jeffreys(3) and by Thomas.(4) If the Manning exponent $n = \frac{2}{3}$ is assumed Eq. (29) becomes

$$S > \frac{g}{4}\ S_c$$

which is the result obtained by Keulegan and Patterson.(5)

It is also possible to derive Vedernikov's formula from Eq. (21). Vedernikov(6,7) starts with the equation

$$S = \frac{a\ V_o^p}{R^{1+\beta}} \tag{30}$$

where R is the hydraulic radius.

The use of this equation would require the substitution of the exponent p for the two exponents on the right-hand side of Eq. (5). However, this change leaves Eq. (21) unaltered.

Solving Eq. (30) for V_o there is obtained

$$V_o = (\frac{S}{a})^{\frac{1}{p}} R^{\frac{1+\beta}{p}}$$

Taking the derivative of this expression, and then dividing this derivativ the expression itself yields

$$\frac{dV_o}{V_o} = \frac{1+\beta}{p} \frac{dR}{R}$$

Since

$$dw = c \frac{dA}{A}$$

it is possible to write

$$\frac{dV_o}{dw} = \frac{1+\beta}{p} \frac{A\,V_o}{R\,c} \frac{dR}{dA}$$

but

$$\frac{dR}{dA} = \frac{d}{dA} (\frac{A}{p}) = \frac{1}{p} (1 - R \frac{dp}{dA})$$

and therefore

$$\frac{dV_o}{dw} = \frac{(1+\beta)\,V_o}{p\,c} (1 - R \frac{dp}{dA})$$

where P is the wetted perimeter.

Letting μ be the velocity of a pulse moving along a forward characte so that

$$\mu = V_o + c$$

c can be eliminated from Eq. (32) with the result

$$\frac{dV_o}{dw} = \frac{(1+\beta)\,V_o}{p\,(\mu - V_o)} (1 - R \frac{dP}{dA})$$

The expression on the right-hand side of Eq. (34) has been called the V nikov number by Powell[8] who states that when this number exceeds o flow is ultra-rapid, roll waves form and the flow cannot be steady. Ve nikov's criterion is, therefore, equivalent to Eq. (21). However, both h been developed here for a uniform distribution of velocity in the channe cross-section. Since Vedernikov introduces the velocity-distribution c ficient into his formula in a way different from that followed by the writ his original derivation, the two criteria would differ somewhat for a no uniform distribution of velocity.

Craya[9] has shown that Eq. (21) is equivalent to

$$u > V_o + c$$

where

$$u = \frac{dQ_o}{dA}$$

is the celerity of Seddon, Q_o being the normal discharge. Eq. (35) can derived by substituting

$$dV_o = \frac{AdQ_o - QodA}{A^z}$$

and

$$dw = c\,\frac{dA}{A}$$

into Eq. (21). If the same substitution is made into Eq. (23) there is obtained

$$u < V_o - c \tag{37}$$

which is the corresponding condition for waves of instability along the backward characteristics.

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REVISED COMPUTATIONS OF A VELOCITY HEAD WEIGHTED VALUE[a]

Discussions by Byron N. Aldridge and T. R. Anand

BYRON N. ALDRIDGE,[1] A. M. ASCE.—The author's main objection to the use of the present method of determining a weighted velocity head, $\dfrac{\sum \frac{K^3}{A^2}}{\frac{(\Sigma K)^3}{(\Sigma A)^2}}$ appears to be the cumbersome figures obtained. Since only the relative values of the numerator and denominator are needed, the work can be reduced considerably by arbitrarily shifting the decimal point in both the area and conveyance columns and making mental calculations of K^3/A^2. The U. S. Geological Survey uses this method in much of its slope-area work. The method is illustrated below with the example used by the authors in their presentation.

		Approximate values				
A_1	K_1	$\frac{K_1}{A_1}$	$(\frac{K_1}{A_1})^2$	$(\frac{K_1}{A_1})^2\ K_1$	Slide rule reading	$\frac{K_1^3}{A_1^2}$
1 ,895	17 5,000	10	100	1,700	149	1,490
176	3,670	2	4	1 2	160	1 6
82.4	935	1	.1	9	122	1 22
						1,492 82
2 ,153.4	17 9,605	8	64	1,150	125	1,250

$$\text{alpha} = \frac{1,492}{1,250} = 1.19$$

The lines through the A and K columns are drawn arbitrarily to obtain small numbers. The decimal can be shifted any amount and need not be shifted the same amount in the area column as it is in the conveyance column.

The columns headed "approximate" can be obtained by rapid mental calculation to give the location of the decimal point for the shifted computations, and need not be recorded.

Where there is a large variation in conveyance it is often more convenient to use decimals in place of whole numbers. In the above example the decimal in the area column could be shifted 2 places instead of 3, which would move

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a. Proc. Paper 2149, September, 1959, by J. M. Lara and K. B. Schroeder.

1. Hydr. Engr., U. S. Geological Survey, Surface Water Branch, Tacoma, Wash.

the decimal one place further to the left in the $\frac{K_1}{A_1}$ column and 2 places fur to the left in the remaining columns.

The value of alpha is the same by either method and is correct even th the absolute values of K^3/A^2 are not known.

When the mental calculations show that $\frac{K^3}{A^2}$ of small areas will not effec the last significant figure of the total it is not necessary to compute $\frac{K^3}{A^2}$ fo these areas. The data that the writer would record on the computation sh for the example is shown below.

A	K	$\frac{K^3}{A^2}$	Alpha
1,895	175,000	14.9	
176	3,670	-	
82.4	935	-	
		14.9	
2,153.4	179,605	12.5	1.19

Velocities in the subsections are computed for the final discharge (whi is usually obtained by weighting the discharge from 2 or 3 subreaches) in same manner as that shown by Lara and Schroeder.

T. R. ANAND,[1] A. M. ASCE.—A study of the "revised" formulas recom mended by the authors shows that these are essentially the same as the "c formula, viz;

$$\frac{v^2}{2g} = \frac{1}{2g} \cdot \frac{\Sigma \frac{K_p^3}{A_p^2}}{\frac{\Sigma K_p^3}{\Sigma A_p^2}} = \frac{1}{2g} \cdot \alpha$$

Where α is the velocity head correction and suffix "p" a portion of the cha This can be proved as follows, starting with the following "new" steps of t author's.

$$V_p = \frac{Q_p}{A_p} \quad ----$$ (2) col. 8, pp. 7

$$h_v = \frac{\Sigma V^2 Q}{Q \cdot 2g} \quad --$$ col. 10, pp. 7

$$= \frac{\Sigma \left(\frac{Q_p}{A_p}\right)^2 Q}{Q \cdot 2g}$$

1. Engr., The Shawinigan Eng. Co., Ltd., Montreal, Quebec.

$$= \frac{\sum \frac{Q_{\hat{p}}^{3}}{A_{\hat{p}}^{2}}}{Q \cdot 2g} \quad --- \qquad (3)$$

$$\text{now} \quad Q_{\hat{p}} = A_{\hat{p}} \cdot V_{\hat{p}}$$

$$= A_{\hat{p}} \times \frac{1.486}{n} R_{\vee}^{2/3} S_{\vee}^{1/2}$$

$$\text{or} \quad Q_{\hat{p}} = K_{\hat{p}} \cdot S_{\vee}^{1/2} \quad ---- \qquad (4)$$

Substituting (4) in (3) we get

$$h_{\hat{v}} = \frac{\sum S_{\vee}^{1/2} \cdot \frac{K_{\hat{p}}^{3}}{A_{\hat{p}}^{2}}}{2g \sum K_{\hat{p}} S_{\vee}^{1/2}} = \frac{1}{2g} \frac{\sum K_{\hat{p}}^{3}/A_{\hat{p}}^{2}}{\sum K_{\hat{p}}}$$

which is the "old" formula given by (1).

As for the convenience and saving in computational work, the writer fails to see how working out value of α from formula (1) in one step can be more cumbersome than filling up a seventeen column table. If velocities and discharges are required in a subdivision, they can be easily worked out using the various values of K_p.

Also, it is objectionable to introduce the conveyance term $K_p = \frac{1.486}{n} AR^{2/3}$ in hydraulic calculations when a similar term $K = 1.486\ AR^{2/3}$ is already well established. This latter term depends solely on channel geometry and has a predictable shape when plotted for various types of channels. Mannings "n" can differ from reach to reach for the same channel and should be used separately.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

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AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

c. Discussion of